PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XL—No. 10

DECEMBER, 1914

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1914

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American Society of Civil Engineers

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ON MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard, George W. Tillson, Nelson P. Lewis, Charles J. Tilden.

ON VALUATION OF PUBLIC UTILITIES: Frederic P. Stearns, Charles S. Churchill, Leonard Metcalf, William G. Raymond, Henry E. Riggs, Jonathan P. Snow, William J. Wilgus.

TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS: Nelson P. Lewis, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, John A. Bensel.

TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, Frank M. Kerr.

ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

ON FLOODS AND FLOOD PREVENTION: C. McD. Townsend, John A. Bensel, T. G. Dabney, C. E. Grunsky, Frank M. Kerr, Morris Knowles, J. B. Lippincott, Daniel W. Mead, John A. Ockerson, Arthur T. Safford, Charles Saville, F. L. Sellew.

TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Robert W. Hunt, George W. Kittredge, C. G. E. Larsson, William McNab, G. J. Ray, F. E. Turneaure, J. E. Willoughby.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.....5913 Columbus.
CABLE ADDRESS.....“Ceas, New York.”

*Elected to fill the vacancy caused by the death of Emil Gerber, Director, on April 16th, 1914.

Vol. XL. DECEMBER, 1914. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

November 18th, 1914.—The meeting was called to order at 8.30 P. M.; Vice-President J. Waldo Smith in the chair; Chas. Warren Hunt, Secretary; and present, also, 186 members and 30 guests.

A paper by S. W. Bowen, M. Am. Soc. C. E., entitled "The Design and Construction of Four Reinforced Concrete Viaducts at Fort Worth, Texas," which had been postponed from the meeting of October 21st, 1914, was presented for discussion.

A paper by J. H. Gandolfo, Assoc. M. Am. Soc. C. E., entitled "The Valuation of Public Utility Property," was presented by the Secretary, who also read a communication on the subject from Alex Dow, M. Am. Soc. C. E. The paper was discussed by Messrs. W. W. Crehore and F. Lavis.

A paper by John W. Alvord, M. Am. Soc. C. E., entitled "Fundamental Principles of Public Utility Valuation," was presented by the

author. The subject was discussed by Messrs. Alexander C. Humphreys, F. Lavis, L. L. Tribus, and Charles R. Harte.

Communications on the subject from Messrs. L. M. Haupt, L. L. Jewel, and Richard T. Dana, were not read on account of the lateness of the hour.

The Secretary announced the following deaths:

EDWARD COOK BURNS, of Jamestown, N. Y., elected Member, July 5th, 1882; died October 29th, 1914.

EMIL KUICHLING, of New York City, elected Member, September 3d, 1884; died November 9th, 1914.

Adjourned.

December 2d, 1914.—The meeting was called to order at 8.30 P. M.; Vice-President Gardner S. Williams in the chair; Chas. Warren Hunt, Secretary; and present, also, 136 members and 21 guests.

The minutes of the meetings of October 21st and November 4th, 1914, were approved as printed in *Proceedings* for November, 1914.

A paper by Minton M. Warren, Jun. Am. Soc. C. E., entitled "Penstock and Surge-Tank Problems" was presented by the author.

A paper by Raymond D. Johnson, Esq., entitled "The Differential Surge Tank" was presented by the author and illustrated with lantern slides.

The two papers were discussed orally by Messrs. John C. Trautwine, Jr., D. L. Webster, Minton M. Warren, and Raymond D. Johnson.

The Secretary presented by title discussions from Messrs. Roy Taylor and Irving P. Church.

The Secretary announced the election of the following candidates on December 2d, 1914:

AS ASSOCIATE MEMBERS

FREDERICK ABELS ALLNER, Baltimore, Md.
CARL ASHLEY, New York City
JOHN LELAND BECTON, Wilmington, N. C.
WILLIAM COOPER CAMPBELL, Columbus, Ga.
JOHN PEALE DE L'EAU, Betteravia, Cal.
GORDON HILDRETH FERNALD, Fitchburg, Mass.
HOWARD WASHINGTON HALL, Bisbee, Ariz.
GEORGE WILLIAM HARRIS, Port Arthur, Ont., Canada
NED ELMER HAWKINS, Youngstown, Ohio
LOUIS RICHARD HOWSON, Chicago, Ill.
EDWARD MELVILLE LYNCH, Glendale, Cal.
HARRY SEYMOUR MCKIBBEN, Warren, Ohio
GEORGE MAGUIRE, Baltimore, Md.
LESLIE WILLIAM MERRIMAN, Philadelphia, Pa.

CHANCEY SPENCER MILLARD, Vancouver, B. C., Canada
HARRY MONETT, Berkeley, Cal.
THOMAS BREW NOLAN, JR., Seattle, Wash.
RALPH WALDO PORTER, Pittsburgh, Pa.
WILLIAM REINHARDT, San Francisco, Cal.
JOHN CLEMENT REMINGTON, JR., Haddonfield, N. J.
PHILIP DURKEE RICE, San José, Cal.
ROBERT EARL RICHARDSON, Longmont, Colo.
JAY LA DREW SOUTHWORTH, Rome, N. Y.
WILLIS TILLMAN SPIVEY, Augusta, Ga.
MORRIS THOMPSON, Tampa, Fla.
LESSLIE RIELLE THOMSON, Montreal, Que., Canada
WILLIAM JACY TITUS, Indianapolis, Ind.
ROBERT OLANDO WHITEAKER, San Antonio, Tex.
JOSEPH FREMONT WILBER, Boston, Mass.
ADAM ZIEGLER, Tenafly, N. J.

AS ASSOCIATE

WILLIAM GREENE STONE, Utica, N. Y.

AS JUNIORS

WALTER SEIGFREID ANDERSON, Chicago, Ill.
ARTHUR JEROME BARZAGHI, New York City
GUY HERSEY BISHOP, Coon Rapids, Iowa
HOWARD FOSTER CLARK, San Francisco, Cal.
ROBERT MORRIS COPELAND, Kansas City, Mo.
WILLIAM GREENFIELD CORLETT, Oakland, Cal.
FRED WILBUR CROCKER, Portland, Ore.
ROBERT EDWARD DAKIN, New Haven, Conn.
DANIEL TALBOT FINKBEINER, Freeport, Pa.
NASRI SOLOMON FULEIHAN, Jerusalem, Palestine
LEONARD BRUCE GALLAGHER, Norristown, Pa.
HOMER ALLAN LESTER, Stanley, Va.
CONLAW ROBERT LYNCH, Newport, R. I.
KARL ANDREW MILLER, Meadville, Pa.
BURKE BROCKWAY ROBERTS, Cleveland, Ohio
LOUIS CLEMENT WAGNER, Houston, Tex.
JOHN MALCOLM WALLER, Kansas City, Mo.
CHARLES SMITH WHITNEY, New York City

The Secretary announced the transfer of the following candidates on December 2d, 1914:

FROM ASSOCIATE MEMBER TO MEMBER

DAVID REGINALD COOPER, New York City
CLARKE NIGHTINGALE KAST, Seattle, Wash.

FRANCIS FIELDING LONGLEY, New York City
RALPH DANFORD THOMAS, Minneapolis, Minn.
ERNEST WOODBURY WIGGIN, New Haven, Conn.

FROM JUNIOR TO ASSOCIATE MEMBER

ERNEST OSWALD BILLWILLER, Stockton, Cal.
JOHN JAMES CLARK, St. Louis, Mo.
HOWARD THOMPSON CRITCHLOW, Harrisburg, Pa.
WARREN ELLIS DAY, Salt Lake City, Utah
JOHN WILLIAM HORTON, Sacramento, Cal.
BENJAMIN EARL JONES, Helena, Mont.
LEWIS THAYER KNISKERN, New York City
ALBERT AUGUST KRIEGER, Louisville, Ky.
ALEXANDER MISCHKE, Warsaw, Russia
VINCENT ROBERTS, New York City
TAOYUH CLARANCE SUN, Shanghai, China
CARL FELIX KILL-MAR VON BLÜCHER, Corpus Christi, Tex.
CHARLES WILLIAM WOODRUFF, Portland, Ore.

The Secretary announced the following deaths:

EDMUND JOB STEERE, of Providence, R. I., elected Member, October 1st, 1913; died July 18th, 1914.

LUCIAN ARNOLD TAYLOR, of Worcester, Mass., elected Member, November 4th, 1891; died November 19th, 1914.

Adjourned.

December 16th, 1914.—Because of the necessity of going to press with this number of *Proceedings* in advance of this meeting, the publication of its minutes must be deferred until January, 1915. Two papers have been set down for discussion: "Submerged Pipe Work at Portland, Oregon," by D. D. Clarke, M. Am. Soc. C. E.; and "History of Little Rock Junction Railway Bridge of the St. Louis, Iron Mountain and Southern Railway Company, Over the Arkansas River, at Little Rock, Arkansas, 1883-1914," by C. E. Smith, M. Am. Soc. C. E.

OF THE BOARD OF DIRECTION

(Abstract)

December 2d, 1914.—The Board met at 3.30 P. M.; President McDonald in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Cummings, Edwards, Endicott, Hodge, Leonard, Loweth, Montfort, Smith, Swain, Tuttle, and Williams.

A Committee was appointed to draft the Annual Report of the Board of Direction.

The Presidents of the thirteen Local Associations of Members of the Society having been invited to meet at about the time of the Annual Meeting for the purpose of discussing matters connected with those Associations so that there might be uniformity of action and a better understanding of their functions, the following was adopted:

"In view of the importance of this particular occasion, mileage will be allowed to the President of each Local Association (or his representative) who attends this Special Meeting."

Charles Warren Hunt was appointed one of the four representatives of this Society on the John Fritz Medal Board of Award to serve four years to fill the vacancy caused by the expiration of his term.

The resignations of 2 Members, 2 Associate Members, and 1 Associate were accepted to take effect December 31st, 1914, and the resignation of 1 Associate Member was accepted to take effect immediately.

Ballots for membership were canvassed, resulting in the election of 30 Associate Members, 1 Associate, 18 Juniors, and the transfer of 13 Juniors to the grade of Associate Member.

Five Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business was transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

January 6th, 1915.—8.30 P. M.—This will be a regular business meeting. A paper by Samuel Tobias Wagner, M. Am. Soc. C. E., entitled "The Water-Proofing of Solid Steel-Floor Railroad Bridges," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

February 3d, 1915.—8.30 P. M.—A regular business meeting will be held, and a paper by R. C. Strachan, M. Am. Soc. C. E., entitled "Nomographic Solutions for Formulas of Various Types," will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL MEETING

The Sixty-second Annual Meeting will be held at the Society House, on Wednesday and Thursday, January 20th and 21st, 1915.

COMMITTEE OF ARRANGEMENTS.

J. S. LANGTHORN,
WALTER H. GAHAGAN, JOHN HAYS MYERS,
J. H. GRANBERY, CHARLES WARREN HUNT.

The Business Meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be presented, officers for the ensuing year elected, members of the Nominating Committee appointed. Reports of Special Committees presented for discussion, and other business transacted.

An excursion will be arranged for the afternoon of Wednesday, and in the evening there will be a reception, with dancing, at the Hotel Astor. Thursday will be devoted to an all-day excursion, the details of which will be announced later; and on Thursday evening there will be a Smoker at the Society House.

SPECIAL MEETINGS

Meetings, for discussion of the Report of the Special Committee on Materials for Road Construction, will be held at 10 A. M. and 2 P. M. on Friday, January 22d, 1915 (the day following the close of the Annual Meeting of the Society).

PROGRESS REPORTS OF SPECIAL COMMITTEES

Progress Reports of the Special Committees on Materials for Road Construction, on Steel Columns and Struts, and on Conditions of Employment of, and Compensation of, Civil Engineers, are printed in this number of *Proceedings* with the Papers and Discussions.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

In making a search it sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911)

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Albany Hotel.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

November 21st, 1914.—The meeting was called to order; President Vincent in the chair; Roger W. Toll, Secretary; and present, also, 21 members and guests.

A letter was read from the Secretary of the Society, inviting the Association to send a representative to attend a meeting of the Presidents of the Local Associations to be held in New York on January 20th-21st, 1915. On motion, the invitation was accepted, and the Executive Committee was instructed to take such action in the matter as it shall decide.

The Secretary announced that no meeting of the Association had been held during October.

A paper by H. W. Cowan, M. Am. Soc. C. E., on "The Reconstruction of the Union Depot in the City of Denver", was presented by the author, and the subject was generally discussed.

A vote of thanks was tendered Mr. Cowan for his interesting paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club, Atlanta, Ga.

At the meeting of the Association on December 29th, 1913, the new Chairman, John Ruddle, M. Am. Soc. C. E., was installed, and Messrs. Park A. Dallis and G. R. Solomon were appointed members of the Executive Committee. T. P. Branch, Assoc. M. Am. Soc. C. E., was elected Secretary.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Louisiana Association

The Louisiana Association of Members of the American Society of Civil Engineers (New Orleans, La.) has been organized with the following officers: Frank M. Kerr, President; J. F. Coleman and W. B. Gregory, Vice-Presidents; A. M. N. Blamphin, Treasurer; and L. C. Datz, Secretary.

Northwestern Association

At its meeting of November 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Northwestern Association of Members of the American Society of Civil Engineers (St. Paul and Minneapolis, Minn.).

Philadelphia Association

The Philadelphia Association of Members of the American Society of Civil Engineers was organized on December 22d, 1913. The meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

(Abstract of Minutes of Meetings)

October 5th, 1914.—President George S. Webster in the chair; W. L. Stevenson, Secretary; and present, also, 70 members, and a number of guests.

The Annual Reports of the Secretary and Treasurer were read and approved.

The principles embodied in the proposed plan for the federation of engineers of Philadelphia and vicinity, were approved by the Association.

The draft of an Act for Licensing Engineers in Pennsylvania, prepared by the Engineers' Commission in order to obtain expressions of opinion, was discussed, and the following motion was carried:

"That it is the sense of this meeting that it is undesirable and unnecessary to license engineers."

In order that the subject might be presented to the Association in the most complete manner before final action, the President was authorized to appoint two committees of three; one to submit arguments for, and the other against, the licensing of engineers, at an adjourned meeting to be held one week later.

Ballots were canvassed, and the following officers were elected for the ensuing year: President, Richard L. Humphrey; Vice-Presidents, F. Herbert Snow and William Hunter; Directors, John Sterling Deans, John W. Ledoux, Edgar Marburg, and H. S. Smith; Treasurer, S. M. Swaab; Secretary, W. L. Stevenson.

Hunter McDonald, President, Am. Soc. C. E., addressed the meeting on the subjects of the Federation of Engineering Societies and on the Licensing of Engineers.

Adjourned.

October 12th, 1914.—President Richard L. Humphrey in the chair; W. L. Stevenson, Secretary; and present, also, 27 members and guests.

Messrs. John A. Vogleson, Chairman, and F. W. Abbott, of the Committee to submit arguments in favor of the licensing of engineers (of which the third member was Harrison Souder, M. Am. Soc. C. E.), addressed the meeting.

Edgar Marburg, M. Am. Soc. C. E., Chairman of the Committee to submit arguments against the licensing of engineers, presented a report signed by himself and Messrs. Henry H. Quimby and S. M. Swaab, the other members of the Committee.

After a discussion of the two reports, the following motion was passed:

"That the action of the previous meeting be reaffirmed."

The President was authorized to appoint a Committee on Licensing Engineers to represent the Association at the hearing of the Engineers' Commission, to keep in touch with the movement, and, if deemed advisable, to appear before any legislative committees.

(In conformity with this action, President Humphrey appointed the following Committee: Edgar Marburg, Chairman, John Sterling Deans, John W. Ledoux, H. R. Leonard, Henry H. Quimby, S. M. Swaab, Edward B. Temple, Samuel Tobias Wagner, and Richard L. Humphrey, *ex-officio*. This Committee amplified the report of the Committee of Three which had submitted arguments against the licensing of engineers. This report was presented by the Committee to the Engineers' Commission at the public hearing in Philadelphia, on October 19th, 1914, and the Commission requested the Committee to submit a supplementary report enlarging on the constructive features of the original report.)

Adjourned.

October 26th, 1914.—President Richard L. Humphrey in the chair; W. L. Stevenson, Secretary; and present, also, 35 members.

The Committee on Licensing Engineers submitted a Supplementary Report as requested by the Engineers' Commission.

This report recommended the creation of a State Department of Public Works which shall be in charge of (1) the Civil Service Examination System, and (2) a Supervisory System of Plans and Inspection of Engineering Works, and in which shall be centralized such other matters relating to public engineering activities as may seem proper.

After discussion, the following motion was passed:

"That this meeting endorse the principles embodied in the supplementary report of the Committee on Licensing Engineers and approve its transmission to the Engineers' Commission of Pennsylvania."

Adjourned.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized, with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

St. Louis Association

At its meeting of October 7th, 1914, the Board of Direction considered and approved the proposed Constitution of the St. Louis Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, J. A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, F. J. Jonah; Secretary-Treasurer, Gurdon G. Black. The meetings of the Association are held at the Engineers' Club Auditorium.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year; Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bimonthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, M. Am. Soc. C. E., 515 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, C. S. MacCalla; Vice-President, U. B. Hough; Second Vice-President, Morton Macartney; Secretary-Treasurer, A. D. Butler.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers. The headquarters of the Association is Dallas, Tex.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

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Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

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Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

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Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

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Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

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THE REGULATION OF RIVERS.

By J. L. Van Ornum, M. Am. Soc. C. E. Cloth, 9½ x 6¼ in., illus., 10 + 393 pp. New York and London, McGraw-Hill Book Company, Inc., 1914. $4.00.

In this book, it is stated, the different methods of river regulation and improvements and their particular applicability to the varying conditions of such waterways, are discussed in detail, as well as the fundamental and scientific principles underlying such work, and in order to secure a more complete knowledge of the scientific laws involved, the author has presented quite fully the methods and results of experimental investigations and other propositions. He has assumed, it is said, a prior knowledge of Surveying and Hydraulics, and, therefore, the discussion of such introductory features includes only those topics necessary to an adequate comprehension of the whole subject. Figures, statistics, plans, and operations have been summarized, it is stated, or representative types have been given to illustrate and elucidate the fundamental principles involved. Many references are also given in the text and in foot-notes to books and papers which may be used as collateral reading on the subject. The Chapter headings are: Introduction; Commercial Considerations; General Phenomena; Investigations, Surveys, etc.; Methods of River Improvement; The Principles of Regulation; Works of Channel Contraction; The Protection of Erodible Banks; Dredging; Levees; The Control of the Current; Index.

POLYPHASE CURRENTS.

By Alfred Still. Second Edition, Revised. Cloth, 7¾ x 5 in., illus., 12 + 300 pp. New York, The Macmillan Co.; London, Whittaker & Co., 1914. $1.50.

The author states in the preface to this edition that the scope and viewpoint of the subject have not been changed, the object being to present the principles underlying the operation of polyphase currents in clear and simple terms. The book is intended, it is said, primarily for the student and user who may have to lay out and work out a complete polyphase scheme, exact details of machinery, etc., being omitted. The treatment of the subject is non-mathematical, it is stated, but extensive use has been made of graphical methods. The original matter has been rearranged, rewritten, and some of it entirely omitted as deemed necessary, the assumption still being held that the student is unfamiliar with alternating currents. All the figures and diagrams are said to have been redrawn, and all vector diagrams have been reversed to accord with the internationally agreed convention regarding leading and lagging vectors, and the symbols used to denote certain physical quantities have been changed for the same reason. The Chapter headings are: Elementary Study of Alternating Currents; Self-Induction and Capacity; Polyphase Currents: General Principles and Synchronous Generators; Measurement and Calculation of Power on Polyphase Circuits; Polyphase Transformers; Power Transmission by Polyphase Currents; Polyphase Induction Motors; Synchronous Generators, Frequency Converters, Compensated Induction Motors, and Rotary Converters; Appendix I, On the Relation Between Magnetic Flux and Induced E. M. F. in a Circuit Conveying an Alternating Current; Appendix II, Method of Drawing the Complete Vector Diagram for a Polyphase Induction Motor from Three Sets of Measurements Made on the Machine; Index.

SURGE TANK PROBLEMS:

An Investigation of Surge Tank Regulation Determining by Graphical and Analytical Methods Proper Solutions of Problems Created by Long Pipe Lines. By Franz Prasil. Authorized Translation by E. R. Weinmann and D. R. Cooper, Assoc. M. Am. Soc. C. E. Paper, 12 x 9 in., illus., 27 pp. Toronto, The Canadian Engineer; New York, E. R. Weinmann, 1914. 50 cents. (Donated by E. R. Weinmann.)

This paper includes, it is stated, a discussion of the problems leading to the determination of the dimensions and the development of methods, partly analytical and partly graphical, involved in the solution of these problems, on the assumption

* Unless otherwise specified, books in this list have been donated by the publishers.

that the main conduit from the intake to the surge tank consists of a pipe or tunnel running full under pressure. The translators have added a short bibliography on surge tank regulation which is printed at the end of the paper. The Contents are: Introduction; Derivation of Principal Equations; Special Cases; Investigation of the Influence of $h = U.V^2$; Conclusions; Appendix A, Surge Tank with Variable Cross-Section; Appendix B, Simplified and Approximate Formula for the Determination of Surge Tanks with Constant Cross-Section; Bibliography.

STRUCTURAL DESIGN:

Volume II, Design of Simple Structures. By Horace R. Thayer, Assoc. M. Am. Soc. C. E. Cloth, 9 x 6 in., illus., 9 + 495 pp. New York, D. Van Nostrand Company, 1914.

In the first volume of this series, the author discussed the elements of structural design. In this, the second volume, which presupposes, it is stated, a knowledge of mechanics, stresses, and the mathematics on which they depend, he has considered the design of such simple structures as beams, girders, viaducts, trusses, buildings, stand-pipes, and elevated tanks. An attempt, it is said, has been made to discuss these subjects from both the theoretical and practical standpoints, to which end a short bibliography of related works is given at the end of many of the subjects discussed, and many concrete examples are included. The Contents are: Beam Bridges, Plate Girders; Steel Viaducts and Elevated Railroads; Simple Riveted Truss Bridges; Simple Pin-Connected Bridges; Steel Mill Buildings; High Office Buildings; Miscellaneous Building Work; Stand-Pipes and Elevated Tanks; Index.

PRACTICAL RATE MAKING AND APPRAISEMENT.

By William D. Marks. Cloth, 7½ x 5 in., illus., 269 pp. South Hadley, Mass., Office, President's House, 1914. $2.00. (Donated by Jeannette Marks.)

The author's aim in this work, it is stated, is to be entirely practical and as brief as a discussion of the subject will permit. He has made many valuation reports and appraisements, it is said, and in this book gives a recapitulation of the more extended ones, including their objects, costs, and the local peculiarities connected with them. A few of his briefer reports are reproduced in full, and these are followed by a general discussion of the principles involved in the rate making and appraisement, with practical examples of their application. The Contents are: General Considerations; Report Upon Gas Rates and Prices for the City of Spokane, 1913; Report Upon the Fair and Reasonable Price of a 6.6 Ampere Magnetite Arc Light per Year, Minneapolis, 1911; Quantity Rates for Electricity; Methods of Appraisement; Present Value, Structural Cost of Reproduction, Original Cost, Structural Cost of Reduplication; Overhead Charges; Depreciation; Comparative Appraising; Market Value, English Going Value; Franchise Value; Present Value of Prospective Profits, Goodwill; Development Expense, Going Value, Intangible Values, Options; Books and Accounts, Operating Expenses and Income; The Law of Demand for Electricity, Reduction of Price Increases Profits; Gas Sales per Capita, Verification of Law of Demand; The London Sliding Scale, Marks' Dividing Scale; Capitalization, Assets and Profits of Massachusetts Gas Companies.

SURVEYING MANUAL

Designed for the Use of First-Year Students in Surveying and Especially for the Use of Non-Civil Engineering Students. By Howard Chapin Ives, Assoc. M. Am. Soc. C. E. Morocco, 7 x 4½ in., illus., 8 + 296 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1914. $2.25.

The preface states that although this Manual was designed as a field guide for first-year students in surveying, it is particularly adapted to the needs of mechanical, electrical, and chemical engineers and of students in agriculture and architecture as well as those in secondary schools. The subject-matter includes descriptions of the various instruments used in surveying and the methods of conducting surveys for various purposes, which are followed by problems relating to the subjects discussed, worked out in detail. Sample field notes reproduced at the same scale as those contained in regular field books and a set of tables sufficient for the ordinary work of surveying, are also included. The Chapter headings are: General Instructions; The Chain and Tape; The Level; The Compass; The Transit; Topographical Surveying; The Plane Table and the Sextant; The Aneroid Barometer and the Planimeter; Railroad Curves; The United States System of Laying Out the Public Lands; Computations; Plotting; Latitude, Longitude, and Azimuth; Large Surveys; Tables; Index.

CONCRETE-STEEL CONSTRUCTION:

Part I—Buildings: A Treatise Upon the Elementary Principles of Design and Execution of Reinforced Concrete Work in Buildings. By Henry T. Eddy, and C. A. P. Turner, M. Am. Soc. C. E. Cloth, 9½ x 6½ in., illus., 15 + 438 pp. Minneapolis, 1914. $6.00. (Donated by C. A. P. Turner.)

As stated in the secondary title, this volume is devoted to a discussion of the elementary principles and practical problems presented by construction work in reinforced concrete. An endeavor has been made, it is stated, to bring out the mechanics of reinforced concrete and to discuss in detail the questions of relative arrangement and disposition of materials in accordance with fixed principles of physics and mechanics, in order that rational rules may be more generally adopted for the safe and economic design of this type of construction. Patented as well as unpatented types are included in order, it is said, that the engineer may familiarize himself with the analysis of the former as well as that of the latter. The requirements of economy in the arrangement and dissemination of the reinforcement are also said to be discussed in considerable detail. The Chapter headings are: Introductory; General Types of Concrete Floor Construction; Beams; Beam Action and Slab Action Compared Through Application of the Laws of Bond Shear and the Theory of Work; Theory of Flat Slabs; Computed Stresses and Deflections Verified by Tests; Moments in Two-Way and Four-Way Flat Slabs; Reinforced Concrete Columns; Foundations; Elements of Economic Construction and Cost of Reinforced Concrete Work; Fireproof Properties of Concrete, etc.; Protection of Steel and Iron from Corrosion by Portland Cement, etc.; Floor Finish, etc.; Artistic and Commercially Practicable Concrete Surface Finishes; The Execution of Work, etc.; Index.

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Zoölogy for High Schools and Colleges. By A. S. Packard. Eleventh Edition, Revised. New York, 1904.

American Society for Testing Materials: Year Book, 1914, Containing the Standard Specifications. Edited by the Secretary-Treasurer, University of Pennsylvania. Philadelphia, 1914.

Modern City Planning and Maintenance. By Frank Koester. New York, 1914.

Principles of City Land Values. By Richard M. Hurd. Third Edition. New York, 1911.

Horse, Truck, and Tractor: The Coming of Cheaper Power for City and Farm. By Herbert N. Casson, Rollin W. Hutchinson, Jr., and L. W. Ellis. Chicago, 1913.

Pumping by Compressed Air. By Edmund M. Ivens. New York and London, 1914.

Fire Protection in Buildings: A Practical Treatise for Engineers, Architects, Surveyors, and Property Owners. By Harold G. Holt. London, 1913.

Interstate Commerce Commission Reports; Vol. 25–28: Decisions of the Interstate Commerce Commission of the United States, October, 1912, to January, 1914. Washington, 1912–14.

Railroad Construction, Theory and Practice: A Text-Book for the Use of Students in Colleges and Technical Schools. By Walter Loring Webb. Fifth Edition, Revised and Enlarged. New York and London, 1913.

American Handbook for Electrical Engineers: A Reference Book for Practicing Engineers and Students of Engineering. Comp. by a Staff of Specialists, Harold Pender, Editor-in-Chief. New York and London, 1914.

Das Wasser: Seine Gewinnung, Verwendung und Beseitigung mit Besonderer Berücksichtigung der Flussverunreinigung. Von Ferdinand Fischer. Leipzig, 1914.

Schwarzpulver und Sprengsalpeter. Von Richard Escales. Zweite völlig umgearbeitete und erweiterte Auflage. Leipzig, 1914.

International Engineering Congress (Glasgow, 1901): The Proceedings of Section 8: Gas. Edited by J. W. Helps. London.

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RESIGNATIONS

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	Date of Resignation.
THON, GEORGE LOUIS	Nov. 4, 1914

ASSOCIATE

AUCHINCLOSS, JOHN WINTHROP	Nov. 4, 1914

JUNIOR

BREWER, WILLARD SEYMOUR	Nov. 13, 1914

DEATHS

BURNS, EDWARD COOK. Elected Member, July 5th, 1882; died October 29th, 1914.

KUICHLING, EMIL. Elected Member, September 3d, 1884; died November 9th, 1914.

TAYLOR, LUCIAN ARNOLD. Elected Member, November 4th, 1891; died November 19th, 1914.

Total Membership of the Society, December 3d, 1914, 7 683.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(November 3d to December 1st, 1914)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Institute Indicator*, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 10c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *Railway Age Gazette*, Mechanical Edition, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer*, Scranton, Pa., 25c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
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(62) *Steel and Iron*, Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
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(79) *Forscherarbeiten*, Vienna, Austria.
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(82) *Mining and Engineering World*, Chicago, Ill., 10c.
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(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
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(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
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(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
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(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.
(109) *Journal*, Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal*, Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas*, San Francisco, Cal., 25c.

LIST OF ARTICLES

Bridges.

Opening Bridges over the Nile at Cairo.* Arthur James Knowles. (63) Vol. 196.
Renewal of Large-Span Girders in Southern India. Percy Rothera. (63) Vol. 196.
Bridge Substructures.* John W. Doty. (5) Vol. 28, Pt. 1.
St. Lawrence River Bridge. P. B. Motley. (5) Vol. 28, Pt. 1.
City and County Highway Drainage. (Culverts.) Frank Reed. (60) Nov.
Use and Development of the Ponton Equipage in the United States Army, with Special Reference to the Civil War. M. J. McDonough and P. S. Bond. (100) Nov.
The Colorado Street Bridge.* (60) Nov.

*Illustrated.

Bridges—(Continued).

Design, Construction and Detailed Labor Costs of the Substructure of the Double-Leaf Trunnion Bascule Bridge at Chicago Ave., Chicago, Ill.* Carl O. Johnson. (86) Nov. 4.
Two Striking Steel Bridges on the Lake Erie & Eastern R. R.* (13) Nov. 5.
Concrete Viaduct with 50-Ft. Walls.* (13) Nov. 5.
Concrete Highway Bridge Construction as Standardized by Iowa Commission.* C. B. McCullough. (14) Nov. 7.
Main Pedestals, Quebec Bridge.* H. P. Borden. (96) Nov. 12.
The Hoang Ho Bridge, China.* (19) Nov. 14.
Ultimate Strength of Carbon and Nickel-Steel Models of Quebec Bridge Members; Tension Tests of Reinforced Steel Plates and of Alternating Stress Members, and Compression Tests of Four Struts.* (14) Nov. 14.
Large Trussed Concrete Bridge, Structure of 182-Foot Span, with Parabolic Reinforced Concrete Compression Chord, Believed to be Longest of its Kind.* (14) Nov. 14.
Design and Construction of the Lower Ganges Bridge in India.* (86) Nov. 18.
Slate Creek Bridge, Building a Concrete Arch Bridge 56 Mi. from Rail.* Jean H. Knox. (13) Nov. 19.
Erecting the Snake River Viaduct: O.-W. R. R. & Nav. Co.* W. J. Howard. (13) Nov. 19.
The World's Longest Bascule Bridge.* Herbert T. Wade. (46) Nov. 21.
Design and Construction of the San Jacinto St. Reinforced Concrete Bridge, Houston, Texas.* (86) Nov. 25.
Shifting the Milwaukee River Drawbridge; C. & N. W. Ry.* (13) Nov. 26.
Conveying Concreting Material by Belts, at the Hell Gate Bridge.* (13) Nov. 26.
Main Span of Detroit-Superior Bridge over Cuyahoga River at Cleveland.* (14) Nov. 28.
Incasing Steel on Rock Island Bridge at Chicago.* (14) Nov. 28.
Schleppbahnbrücke aus Eisenbeton in Pöchlarn.* Hans Schäfer. (80) Sept. 26.

Electrical.

The Fire Hazard in Turbo-Generators. G. S. Lawler. (55) Vol. 35.
Discussion on High Tension Insulators (before the Canadian Soc. of Civ. Engrs.).* (5) Vol. 28, Pt. 1.
Automatic Voltage Regulators.* Chas. C. Garrard. (73) Serial beginning Oct. 30.
Reinforced Concrete Poles for San Francisco Municipal Railway.* John S. Martin. (67) Nov.
The Corona Produced by Continuous Potentials.* Stanley P. Farwell. (42) Nov.
Protective Reactors for Feeder Circuits of Large City Power Systems.* James Lyman, Leslie L. Perry, and A. M. Rossman. (42) Nov.
Use of Reactance with Synchronous Converters an Insurance to Continuity of Service and a Protection to Apparatus.* J. L. McK. Yardley. (42) Nov.
La Salle Plant Saves $45 000 Annually.* Thomas Wilson. (64) Nov. 3.
The Government Electric Power Station of Bangkok.* Francis B. Shaw. (26) Serial beginning Nov. 6.
On Atmosphere Refraction and Its Bearing on the Transmission of Electro-Magnetic Waves Round the Earth's Surface.* J. A. Fleming. (Abstract of paper read before the Physical Soc.) (73) Nov. 6.
The Telephone Plant with Relation to Fires.* W. D. Scott and G. P. Nock. (111) Nov. 7.
Air-Break Switches. W. P. Hammond. (From the *General Electric Review.*) (19) Nov. 7.
Seaview Hospital Plant.* A. D. Blake. (64) Nov. 10.
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INSTITUTED 1852

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GROUTED CUT-OFF FOR THE ESTACADA DAM.

By Harold A. Rands, Assoc. M. Am. Soc. C. E.

To be Presented February 18th, 1914.

Early in 1912, the Portland Railway, Light and Power Company, which is the principal public service corporation of Portland, Ore., completed what is known as the Estacada Hydro-Electric Development. This work, which includes an Ambursen dam, 90 ft. high, with power-house and transmission line, has already been described in several technical journals, and to redescribe these features is not the object of this paper. Inasmuch, however, as the treatment of the foundation material by the grouting process is the greatest thus far attempted, it is believed that a setting forth of this particular feature of the work will prove of interest to many engineers.

The location of the Estacada or River Mill plant, as it is sometimes called, is some 25 miles southeast from Portland, in the extreme western foot-hills of the Cascade Range, at the point where the Clackamas River, issuing from its long and tortuous canyon, begins its course through a rolling alluvial plain. The water-shed above the dam is approximately 800 sq. miles, and though the minimum flow for a short time in the dry months of August and September may fall to 700

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

sec-ft., the average, of course, is much higher, and at times of extreme flood, like that of November, 1909, may rise to 40 000 sec-ft.

Previous Experience.—The Company was not without experience in construction work in these regions. For years it had operated a hydro-electric plant at Oregon City, where the Willamette River drops over a basaltic ledge with a fall of 40 ft. In 1906 it completed the Cazadero Station, 3 miles above the new Estacada plant. The operating head at this station is 130 ft., but the dam, which is 1¾ miles up the river from the plant, raises the water only 45 ft., the remainder being obtained by the flume, ditch, and dike by which the water is conveyed to the forebay. This dam is of logs, and though there is some seepage passing through and around it, this never has been excessive, and Mr. T. W. Sullivan, Hydraulic Engineer for the Company, who completed the work after it had been acquired in an incompleted state from other interests, states that no springs have at any time appeared, either in the bed of the river or in the vertical sides of the gorge, as a result of the dam.

In 1907 the Company began work preliminary to the construction of a dam about 150 ft. high at the head of slack water 2 miles above the Cazadero Dam. Here, under the supervision of S. C. Hulse, Assoc. M. Am. Soc. C. E., extensive borings were put down and tests of various kinds were made to determine the feasibility of erecting so high a structure on the formation found. These borings disclosed rock of so porous a nature that even a moderate pressure applied to the casings of the drill holes produced a leakage of many gallons per minute. Pressure applied to one hole would also cause the water to rise in other holes some distance away. The engineers had the benefit of the knowledge gained by the above construction and experience when in 1909 the Estacada site became the property of the Company.

Preliminary Borings.—At the site proper the river flows through a gorge of lava conglomerate, the sides of which are nearly vertical and from 50 to 70 ft. high. They are capped with a sloping over-burden of gravel, small boulders, and clay from 2 to 20 ft. deep. Preliminary borings, ranging in depth from 60 to 250 ft. and aggregating 2 500 ft. of drilling, were put down on both sides of the river for a distance of several hundred feet up and down stream, as well as on the island which later became a part of the dam.

That the general condition of the Estacada site would be similar to that at the site of the proposed 150-ft. dam previously mentioned was to be anticipated by one having a general knowledge of this region, of its geology, and of the great lava flow which now covers it. Concerning this flow the following is quoted from the Encyclopedia Britannica:

"This is probably the grandest lava-flow known to geology, covering as it does an area of about 200 000 square miles. Commencing in middle California as separate streams, in northern California it becomes a flood, completely mantling the smaller and flowing around the greater inequalities. In northern Oregon and Washington it becomes an absolutely universal flood, beneath which the whole original face of the country, with its hills and dales, mountains and valleys, lies buried several thousand feet. It covers the greater portion of northern California and north-western Nevada, nearly the whole of Oregon, Washington, and Idaho, and runs far into British Columbia on the north. The average thickness is probably not far from 2 000 feet, and the greatest (shown where the Columbia, Des Chutes, Snake, Salmon, and other rivers cut through it) about 4 000 feet. To produce this many successive flows took place, and a very long period of time must have elapsed during which the volcanic actions were going on."

Notwithstanding the universality of this great flood, the formations along those streams flowing westward from the Cascade Range with a rapid descent to the sea, as a rule, present less secure banks than do those traversing the high undulating plateau to the east, as even a casual observer who is familiar with the Snake or Des Chutes River Canyons, with their rim rock and columnar-like structure, may have noted. Concerning this western slope formation Mr. J. S. Diller, of the U. S. Geological Survey, in a report made in 1909 covering the site of the proposed 150-ft., Clackamas River Dam, expressed himself as follows:

"The rocks of this canyon wall are of four forms; volcanic breccia, lava sheets, volcanic dikes and terrace gravels, the volcanic breccia being most abundant.

"The conditions that confront the engineer along the Clackamas river in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade range from the Columbia river in Oregon to the Feather river in California, one of the most important power belts in the United States, and the successful solution of the problem it presents at one point will greatly facilitate the work elsewhere."

To the above, and possibly of greater interest to the engineer, may be added a statement from one, who, from an engineering standpoint, has probably a first-hand knowledge of the entire region second to no one. Certainly, in terseness of language and clearness of expression it leaves nothing to be desired. John F. Stevens, M. Am. Soc. C. E., in a report on the Estacada Dam Site, expressed himself as follows:

"The site of the proposed works lies in the west slope of the Cascade range, and this range is well known to be, geologically taken, as of volcanic origin. Go where we may, for hundreds of miles North or South, we will find the upper valleys, canyons and river beds filled with what we call volcanic débris. Sometimes this volcanic matter appears as the hardest kind of basalt, sometimes as loose and friable as garden soil and generally occurs in every imaginable character and degree of solidity between the two, and it is not probable that nature has made any exception in this particular case. The limited number of test pits and borings which have been made show clearly that as far as the area they cover no well-defined, extremely hard masses of rock exist—that the rock is soft and shades away into clay, the latter appearing under superficial examination much like a half burned brick. The whole formation to the non-geologically scientific man seems to be a mass of soft conglomerate, and this description is borne out in striking manner by the appearance of the nearly vertical walls of the canyon at the site of the dam and immediately adjacent."

Fig. 1, showing the left channel and vertical face of the island in the natural state, gives a good idea of the appearance of the rock. The pitted and scarred surface of the weathered breccia, with its hard irregular fragments embedded in the weakly cemented matrix of indifferent sand, is well shown. In some holes the formation, as shown by the preliminary borings, seemed to be much the same down to the bottom, and in others ledges of hard rock, from 5 to 20 ft. thick, were penetrated; in some the sandy matrix, most common in surface croppings, would change with depth to an even-textured clay, sufficiently compact to give a fair percentage of core; in several holes, though not among those put down on the site finally selected for the dam, this clay at considerable depth deteriorated to such a consistency that for many feet the diamond bit was replaced with a star bit, with which, rotating as did the diamond bit, very rapid progress was made for many consecutive feet.

The sandy matrix and, to a lesser degree, the clay-like matrix were ground away by the attrition of the drills and the harder fragments of

FIG. 1.—LEFT CHANNEL. WEATHERED BRECCIA IN NATURAL STATE.

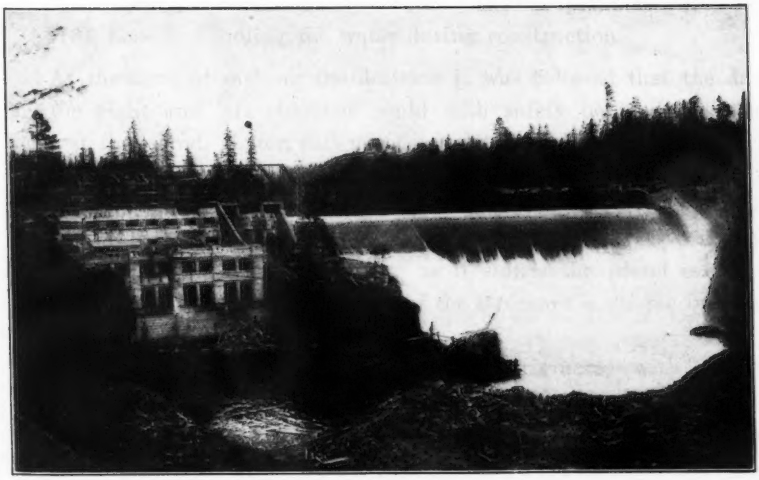


FIG. 2.—DAM, WITH POWER-HOUSE NEARING COMPLETION.

rock so that an average for the diamond bit, using the single-tube core barrel for the Sullivan diamond drill, operated by that Company's expert runners, was slightly less than 50 per cent. The double-tube barrel as used on part of the work gave core amounting to slightly less than 80 per cent. In these borings fragments of wood were passed through at different depths, and in subsequent excavation several stumps, one about 30 in. in diameter, and portions of logs in part petrified and in part in a semi-decayed state were encountered, so that altogether the term "débris" seems most fittingly to describe the material.

The Island Location.—A question constantly being asked by visiting engineers during construction was: Why was the dam located at the island rather than in the narrow gorge some few hundred feet up stream? The reply to this query is as follows:

In the beginning, three sites were under consideration, Site A at the railroad bridge, Site B, 200 ft. below Site A, and Site C, at the island. These are shown on Fig. 3. The reasons which led to the selection of Site C seem to have been:

(1) The soft clay encountered in the preliminary borings already mentioned;

(2) The great length of spillway which was essential, as the crest of the new dam was to be at the same elevation as the low-water tail-race discharge of the Cazadero plant above;

(3) Ease in handling the water during construction.

At the time of making the decision it was believed that the dam in the right and left channels could with safety be made to abut against the island. Later, this was deemed to be taking chances which amounted to gambling with fate, so that over the up-stream side of the island, cut to the slope of the dam, an Ambursen deck, minus most of the steel, was extended. The down-stream slope was also carpeted for a considerable distance, so that as it stands the island is incorporated in the dam, and this portion of the structure is unique in being "a hollow dam" filled with island.

The plan for grouting the foundation originated with Messrs. Sellers and Rippey, of Philadelphia, Consulting Engineers on the work. Their conception of the problem and what they hoped to accomplish is well set forth in a paper by H. V. Schreiber, M. Am. Soc. C. E., and a member of the above firm. This paper was presented

at the Eighth Annual Convention of the National Association of Cement Users. Mr. Schreiber in part said:*

"After a careful study of the conditions disclosed by the investigation it was decided that in order to secure an impermeable foundation it was necessary to practically provide for actually changing the structural character of the underlying formation. To do this Mr. S. Howard Rippey, chief engineer of Messrs. Sellers and Rippey, engineers on the work, recommended that the most promising method would be the solidification of the porous material by the introduction of cement grout under pressure, but that the success of the work must be susceptible of demonstration by actual test before the superstructures should be started. Thorough inquiry failed to disclose any precedent for the use of grout for the general treatment of foundations, although it was found that cavities in limestone rock under the New Croton Dam had been filled with grout, much as a dentist would fill a cavity in a tooth. The grouting method had also been used in filling back of lining walls in tunnels, etc. Notwithstanding the absence of precedent, it was decided to proceed with a grouting scheme and a program was outlined for preventing leakage of water from the reservoir created by the dam, under or around the dam, to the low tail water level below the dam and the experiments which should be made to properly demonstrate the efficacy of the method before the complete development should be undertaken were prescribed. The general idea provided for drilling a double line of holes of an average depth of 50 ft. under the heel of the dam across the entire valley to and under the shore abutments and the subsequent forcing into each of these holes of grout of such consistency as to percolate through the entire substructure.

"It was recognized that experiments would be necessary to determine the proper spacing of the grout holes and their depth, which would insure sufficient diffusion of the grout through the varying material encountered to create a continuous impermeable barrier to prevent seepage of water from the reservoir under the hydrostatic head which would be created by the dam. After drilling the double line of holes, the program contemplated the test of each hole with water pressure, a record being kept of the quantity escaping and the pressure applied to each hole. Upon the completion of the tests, the cement grout was to be pumped into the holes under pressure and after allowing time for hardening, a third line of holes was to be drilled midway between the first two or outer lines and tested with water pressure. The idea was that if water pressure applied to the centre holes at or slightly above the hydrostatic pressure to which the rock would be subjected by the water in the reservoir after completion of the dam and no appreciable leakage occurred, it would be rea-

* *Engineering Record*, March 23d, 1912, p. 314.

Fig. 3.

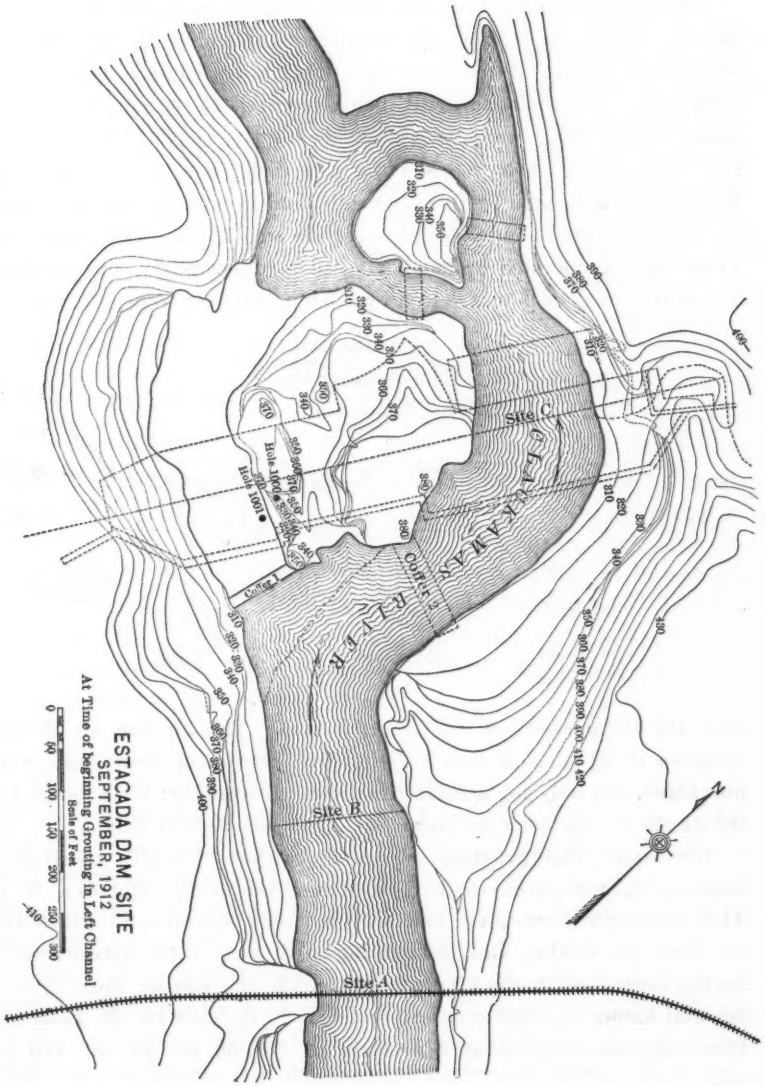

ESTACADA DAM SITE
SEPTEMBER, 1912
At Time of beginning Grouting in Left Channel
Scale of Feet
0 50 100 150 200 250 300
Site C
Site B
Site A
CLACKAMAS RIVER
Hole 1004
Hole 1001
Cofferdam
N

Fig. 3.

sonably certain that the cement grout had proven effective in making the entire foundation impermeable."

The drilling for the grouting included 555 holes, aggregating 34 038 lin. ft. For a short time the Sullivan diamond drills, operated by steam, and also the Davis-Calyx shot drills, operated by electric power, were used. The diamond drills, being required at the site of the proposed 150-ft. dam, were released after putting down 1 119 lin. ft., and from that time to the end the shot drills were used exclusively. These drills are made by the Ingersoll-Rand Company, and those used on this work, with one exception, were the small "Class G-O" machine. These may be operated by hand if speed is of secondary consideration, but on this work each was driven by a 5-h.p. direct-current motor which

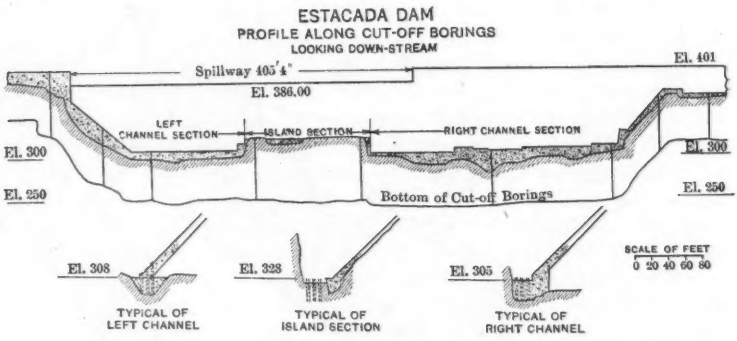


FIG. 4.

gave the bit a speed of 187 rev. per min. The bit had an outside diameter of $2\frac{3}{8}$ in. and gave a $1\frac{1}{2}$-in. core. Seven of these drills were purchased, but only for a short time were all working; the average for the 32 919 ft. put down by them was 13 ft. per 10-hour shift.

Grouting.—The grouting was done with a Canniff air-stirring grouting machine, made by the Tide Water Iron Works, Hoboken, N. J. This wonderfully complete and self-contained machine, in which the air does the mixing and discharging with very little outside work, having been used in the grouting of tunnels and similar work, is now too well known to require description here.* As used on the Estacada Dam, air was supplied at a pressure of 250 lb. per sq. in., and in

* One of these machines was sold later to the U. S. Reclamation Service, and used in grouting the Lahontan Dam of the Truckee-Carson Project. An excellent description of the machine, with drawings, will be found in an article by D. W. Cole, M. Am. Soc. C. E., in *Engineering News*, April 3d, 1913, p. 647.

grouting it was customary to start each hole at a pressure of 25 lb., or whatever was required to start the discharge, and as the hole tightened to increase the pressure, finally ending when at 200 lb. no more grout could be forced in. On this work only neat cement was used. The mixture varied from 1 part cement and 1½ parts water, by volume, to 1 part cement and 15 parts water, the best results seeming to be with 1 part cement and 3, 4, or 5 parts water. In order to prevent the choking of the holes, the scheme was tried of discharging the grout at the bottom of the hole through an inner pipe passing through the cap which sealed the top of the main casing. This method, after a thorough trial, was found of no benefit, and was discontinued.

Arrangement of the Holes.—The plan for the grouting, as proposed by W. L. Fitzgerald, Assoc. M. Am. Soc. C. E., Designing Engineer for the firm of Sellers and Rippey, was that of two lines of holes, 6 ft. apart with 6 ft. between holes in the same line.

These were known as primary holes. To test the effect of the grouting of these primary holes it was proposed that, after allowing a reasonable time for the cement in the primary holes to set, a proving hole should be put down in the center of each square formed by the four primary holes. If this tested tight or reasonably so, that square would be complete. If it did not, additional proving holes would be put down, first on one side and then on the other of the first, so that if all were required in the end there would be four primary and three proving holes in each group.

The Left Channel.

The first grouting done was in the left channel in September, 1910. Across this channel a low coffer-dam, designated on Fig. 3 as Coffer-dam No. 1, had been placed. Back of this a cut-off trench from 8 to 9 ft. deep had been excavated, and through 3-in. casings set with cement 4 ft. in the bottom of this trench, holes were drilled 46 ft. or to a total depth of 50 ft. from the bottom of the cut-off.

The head against the coffer-dam was seldom more than 2 ft., and at times of minimum load at the Cazadero plant, the water receded to the deep-water channel indicated on Fig. 3. Insignificant as this head was, so porous was the ground that it sufficed to cause many of the casings to flow continuously as Artesian wells. Conversely, when pressure was applied to the casings of a number of the holes, leakage

was observed at several points in the river bed, which here was conglomerate bare of gravel, nearly out to the deep-water channel.

Unfortunately, for purposes of comparison, these first holes were tested by a pump taking water from a tank provided with a gauge reading in gallons direct. The pressure gauge was attached to the elbow connecting the water line to the casing of the hole, and the pressures as registered by this and the water taken from the tank were recorded at intervals of 1 min. Tests were made at one-half depth and at full depth for each hole. This method of testing was found to be unsatisfactory, and subsequent tests were made by gravity pressure from an elevated tank also provided with a gauge reading in gallons.

The first attempt to grout was made with the bottom of the cut-off trench bare, but so great was the loss of pressure and the escape of grout through seams in the rock that operations were suspended until the bottom had been protected with a mat of concrete from 2 to 3 ft. thick. At the time of beginning the grouting, though the drills were still working at that point, there had been put down some twenty testing or primary holes. So free was the communication that in grouting one hole it was necessary to cap the neighboring ungrouted holes for some distance in each direction, though it was customary to leave the caps of these holes slightly loose until the air and clear water first forced up gave place to the grout.

Twenty-five holes grouted at this time show results as follows: Average pressure, 19.4 lb.; average leakage, 85.7 gal. per min.; total cement, 167.25 bbl.; average per hole, 6.69 bbl.; most cement in any hole, 40 bbl.

As soon as the grouting was done across the bottom of the left channel, the urgency of getting to work on the power-house and other construction in the right channel made it necessary to blow up the coffer-dam, thus for a time ending operations at this point. This too was unfortunate, for, though it was not known at the time, operations had been commenced in the worst and most porous material encountered on the entire job, and, as future developments were to show, it had to be left but slightly better, apparently, for these efforts.

The Right Channel.

That the construction of the dam proper might proceed with the least possible hindrance from the drills, William H. Cushman, M.

PRIMARY HOLES GROUTED BEFORE CONCRETING

Hole No.	711	712	713	714	715	716	717	718	719	720	
Test Date	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/6'11	4/6'11	4/6'11	
Gall's per Min.	41	100	76	78	96	47	40	75	40	15	
Grout Date	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/10'11	4/21'11	4/24'11	
Bbls. Cement	7.75	5	1.25	25	6.25	6.25	9.25	2.5	1.25	10	
Hole No.	721	722	723	724	725	726	727	728	729	730	Average
Test Date	4/6'11	3/18'11	3/18'11	3/18'11	No	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	
Gall's per Min.	35	75	7	94	Test	4	1	15	30	9	46.2
Grout Date	4/21'11	4/11'11	4/10'11	4/21'11	4/12'11	3/19'11	4/10'11	4/10'11	3/20'11	3/20'11	
Bbls. Cement	3.25	1	3	2	3.75	1	2.75	3.5	1.75	6.25	5.14
Hole No.	511	512	513	514	515	516	517	518	519	520	
Test Date	4/21'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/6'11	4/6'11	4/6'11	
Gall's per Min.	80	31	90	69	95	40	48	55	79	20	
Grout Date	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/26'11	4/30'11	4/10'11	4/17'11	4/24'11	
Bbls. Cement	4.25	1	1	1	5	2.25	4	3.75	4.5	1.75	
Hole No.	521	522	523	524	525	526	527	528	529	530	Average
Test Date	4/6'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	
Gall's per Min.	40	91	93	34	26	93	80	83	13	5	60.75
Grout Date	4/24'11	3/21'11	4/21'11	4/21'11	3/19'11	4/12'11	3/20'11	3/20'11	3/19'11	3/19'11	
Bbls. Cement	5.5	6.5	2	2.5	1.25	3	1	2	3.75	1	2.85

INTERMEDIATE PRIMARY HOLES GROUTED AFTER CONCRETING

Hole No.	711a	712a	714a	715a	718a	719a	720a	721a	722a	
Test Date	6/7'11	6/9'11	6/13'11	6/9'11	6/15'11	6/21'11	6/16'11	6/15'11	6/20'11	
Gall's per Min	22	2	2.5	2	1	2.7	20	40	15	
Grout Date	6/7'11	6/9'11	6/13'11	6/9'11	6/15'11	6/23'11	6/17'11	6/15'11	6/20'11	
Bbls. Cement	11.25	1	1	10.75	1.5	1.25	1	1.75	1.25	
Hole No.	724a	725a	726a	727a	527a	728a	528a	729a	529a	Average
Test Date	6/16'11	6/21'11	6/28'11	6/16'11	7/2'11	6/21'11	7/6'11	6/17'11	6/28'11	
Gall's per Min	2.7	1.2	1.8	3.5	2.5	4.5	1	63	46	13.0
Grout Date	6/17'11	6/23'11	6/28'11	6/17'11	7/2'11	6/23'11	7/6'11	6/23'11	7/2'11	
Bbls. Cement	1.5	1	1.25	1.25	1	1	1.25	1	3	2.72

PROVING HOLES

Hole No.	911 b	913 b	915 b	916 b	918 b	919 b	921 b	923 b	925 b	927 b	928 b	929 b	Average
Test Date	6/25'11	6/9'11	6/7'11	8/16'11	7/17'11	6/20'11	6/25'11	6/20'11	7/5'11	7/3'11	7/13'11	7/8'11	
Gall's per Min	2.5	2	4.2	2.1	1.3	14.6	4	3.5	1.2	3	12.5	44.5	7.95
Grout Date	8/12'11	8/12'11	8/12'11	8/17'11	8/12'11	8/12'11	8/1'11	7/14'11	7/14'11	7/13'11	7/13'11	7/13'11	
Bbls. Cement	4.5	0.75	4	1.25	0.75	1.75	1.50	1	1.25	1.25	1.25	1.5	1.73

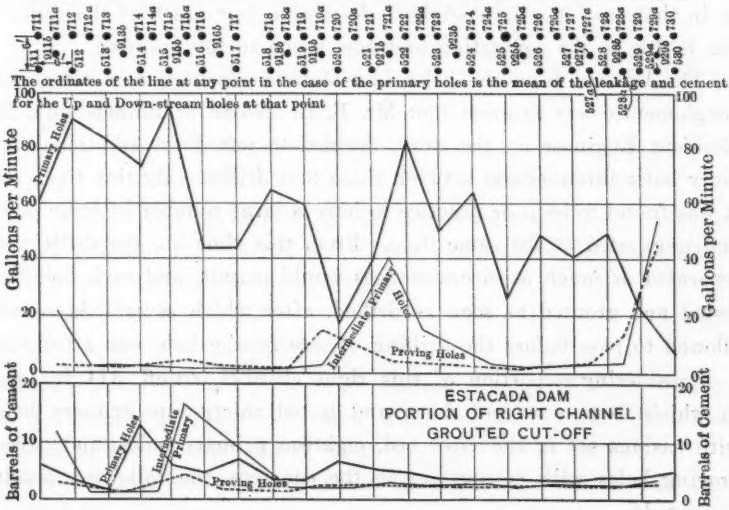


FIG. 5.

Am. Soc. C. E., and representing the consulting engineers, directed that the holes be put down, not through the heel of the dam, as in the left channel, but so as to form a supplementary cut-off immediately up stream therefrom, the plan being to set the casings in the bare conglomerate of the river bottom just back of the cut-off proper, and later, after the grouting and proving were done, to hand-pick a trench to the depth of the casings, namely, 3 ft. or a little deeper. The breccia encountered was sufficiently soft, where speed was not of first consideration, to be removed in this way. This supplementary trench would then be filled with concrete which, bonding with the heel of the dam on one side and with its bottom resting on the curtain-wall supposed to be formed by the grout, would effectually prevent seepage under the dam.

The first grouting was done through casings set in accordance with this plan. However, as in the left channel, the surface material was so lacking in solidity that this method of procedure was not found to be satisfactory, and the plan was modified to the extent of excavating and filling the supplementary cut-off trench before drilling, which was then done through casings set in the concrete. It is apparent that, for purposes of comparison, it is improper to consider the holes tested through casings set in the river bottom with those set in the concrete, as in that case it is impossible to determine how much of the tightening is due to the concreting and how much to the grouting.

The leakage from the holes in which the casings were set in the conglomerate was so great that Mr. F. R. Fisher, of Philadelphia, the Resident Engineer on the work, decided to put down additional primary holes intermediate between those first drilled. By this time, too, it was found to be poor practice to have a large number of holes down and ungrouted at the same time. From this time on, the drills were separated as much as circumstances would permit, and each hole was tested and grouted as soon as drilled, after which several days were allowed to pass before the drilling of any nearby hole was attempted.

Considering a portion of this right channel cut-off, 114 ft. long, in which there were put down and tested thirty-nine primary holes with casings set in the river bed, eighteen primary holes, and twelve proving holes with casings set in the concrete, the following results are noted:

	Gallons per Minute. Maximum.	Minimum.	Average.
39 First primary holes, casings set in river bed....	100	1.0	53.7
18 Second primary holes, casings set in concrete.....	63	1.0	13.0
12 Proving holes, casings set in the concrete.....	44.5	1.2	7.95

The 39 holes first noted, it should be stated, were tested from a tank at Elevation 430, or 46 ft. above the crest of the spillway, and giving a mean head of 182 ft. on the bottom of the hole; the other tests in this, and in the Island and Left Channel Sections following, were made from a tank at Elevation 394, or 7 ft. above the spillway crest, and giving a mean head of 146 ft. on the bottom of the hole.

How much of the tightening, from an average of 54 gal. per min. for the 39 holes drilled and tested before the concrete was placed, to an average of 12.7 gal. per min. for the 18 holes drilled and tested after the concrete was placed, was due to the concrete can never be known. An idea of the relative effect of the concrete and the grout may be deduced by considering an adjoining though somewhat tighter section in which all drilling and testing followed the placing of the concrete. The report for this 54-ft. section is as follows:

	Gallons per Minute. Maximum.	Minimum.	Average.
18 Primary holes, casings set in the concrete.....	17	0.6	4.4
4 Proving holes, casings set in the concrete.....	2.8	0.8	2.2

A comparison of the two tables indicates that but a small percentage of the tightening from 54 to 12.7 gal. per min. in the case of the 114-ft. section was due to the grouting.

Seepage Test.

During construction in the right channel the water above Coffer-dam No. 2 stood at a mean elevation of 317 ft.; behind the coffer-dam, by using pumps, it was held at Elevation 295.

Near the middle of this channel a depression some 30 ft. wide was found in the conglomerate rock. On the cut-off line this was 15 ft.

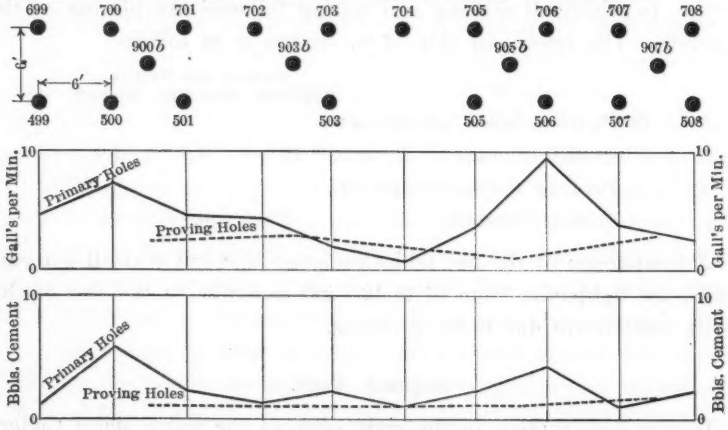
PRIMARY HOLES

Hole No.	699	499	700	500	701	501	702	703	503	
Test Date	6/4'11	6/26'11	6/2'11	5/26'11	5/3'11	6/13'11	5/28'11	5/31'11	5/25'11	
Gall's per Min.	4.4	4.7	2.5	11.9	5.8	3.2	4.3	1.2	2.5	
Grout Date	6/4'11	6/26'11	6/2'11	5/27'11	5/31'11	6/13'11	5/29'11	5/31'11	5/27'11	
Bbls. Cement	0.75	1.25	1	10.5	2.75	1.75	1.25	0.75	4	
Hole No.	704	705	505	706	506	707	507	708	508	Average
Test Date	5/27'11	5/30'11	5/28'11	5/24'11	5/26'11	6/5'11	6/22'11	6/6'11	6/25'11	
Gall's per Min.	0.6	3.2	3.6	17	1.7	3.3	4	1.7	3	4.4
Grout Date	5/27'11	5/30'11	5/29'11	5/25'11	5/26'11	6/5'11	6/22'11	6/7'11	6/25'11	
Bbls. Cement	1	0.75	3.75	7.5	1	1	1.25	1	3.5	2.486

PROVING HOLES

Hole No.	900 *b*	903 *b*	905 *b*	907 *b*	Average
Test Date	6/20'11	6/20'11	6/12'11	6/29'11	
Gall's per Min.	2.5	2.8	.8	2.7	2.2
Grout Date	8/24'11	8/13'11	8/13'11	8/13'11	
Bbls. Cement	1	1	1	1.75	1.187

The ordinates of the line at any point in the case of the primary holes is the mean of the leakage and Cement for the Up and Down-stream holes at that point



ESTACADA DAM
PORTION OF RIGHT CHANNEL
GROUTED CUT-OFF

FIG. 6.

deep, measured from the average level of the surrounding rock, but on the center line of the dam, 80 ft. down stream, this depth had increased to 35 ft. At some earlier age this depression had been worn smooth by the current, but at the time of building the dam it was

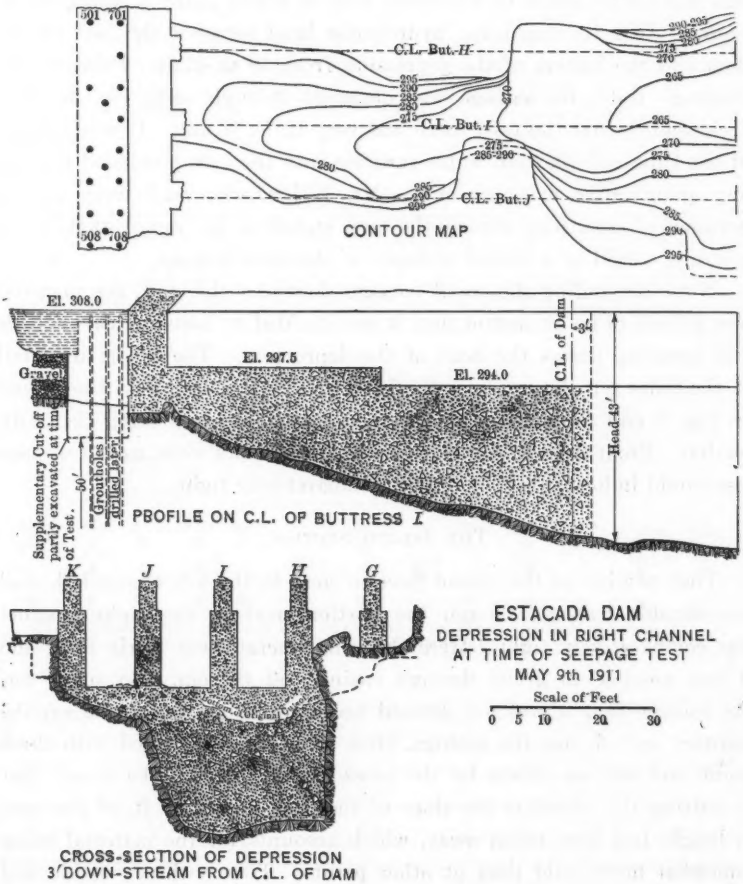


FIG. 7.

filled with loose sand, gravel, and hard boulders. This accumulation was removed, and, as a foundation for the buttresses at this point, mass concrete to a depth of several feet was placed over the entire bottom.

On May 9th, 1911, at which time the concrete had been placed in this gorge to a point 3 ft. below the center line of the dam, and when no drilling and grouting had been done along the cut-off line across the head of the same, the pumps were stopped and the water was allowed to come to Elevation 308, at which point it stood for 6¾ hours. This increased the hydrostatic head between the heel of the dam and the bottom of the depression from 30 to 43 ft. Prior to this "seepage test", no leakage was observed through seams in the bed-rock, and during the test, there was very little, if any. It was raining at the time and the rain water running into the hole precluded making any quantitative measurements. Mr. Fisher, who made very careful personal observations during the test, stated in his report that 2 gal. per min. would be a liberal estimate of the total seepage.

Notwithstanding the small seepage shown by this test, the material was judged to be so erratic that it was decided to continue the drilling and grouting across the head of the depression. The report in detail of the holes put down across the head of this depression will be found on Fig. 6 and summarized in the tabular statement covering the 54-ft. section. From these it will be seen that these holes were, as the seepage test would indicate they should be, comparatively tight.

The Island Section.

That portion of the Island Section next to the left channel showed considerable leakage, though the portion next to the right channel was comparatively tight. Here the conglomerate was fairly firm, and it was possible to grout through casings set therein. So small was the leakage that it was not deemed necessary to hand-pick the supplementary cut-off, but the casings, after grouting, were filled with thick grout and left as shown by the cross-section. It will be noted that in cutting the island to the slope of the deck nearly 50 ft. of the rock in height had been taken away, which accounts for the material being somewhat more solid than at other points. Twenty-three outside and six proving holes put down in this portion show leakage as follows:

		Gallons per Minute.		
		Maximum.	Minimum.	Average.
14	First primary.....	8.5	1.4	4.1
9	Second primary.....	6.6	0.7	2.4
6	Proving	2.3	0.9	1.2

FIRST PRIMARY

Hole No.	666	466	667	467	668	468	669	
Test Date	5/10'11	5/13'11	5/15'11	5/9'11	5/21'11	5/17'11	5/9'11	
Gall's per Min.	2	1.4	4.2	4	4	1.6	2.5	
Grout Date	5/11'11	5/14'11	5/15'11	5/9'11	5/21'11	5/17'11	5/12'11	
Bbls. Cement	1	0.5	1.5	50.25	1	1.25	1.25	
Hole No.	469	670	470	671	471	672	472	Average
Test Date	5/13'11	5/16'11	5/15'11	5/9'11	5/9'11	5/13'11	5/13'11	
Gall's per Min.	3.5	6.5	3.5	8.5	5	4	7	4.1
Grout Date	5/14'11	5/16'11	5/15'11	5/14'11	5/13'11	5/14'11	5/13'11	
Bbls. Cement	1.25	10	2.5	3.5	1	2	19	6.85

SECOND PRIMARY

Hole No.	666 *a*	466 *a*	667 *a*	467 *a*	468 *a*	670 *a*	470 *a*	671 *a*	471 *a*	Average
Test Date	5/9'11	5/20'11	5/24'11	5/22'11	5/19'11	5/20'11	5/18'11	5/16'11	5/18'11	
Gall's per Min.	0.7	0.9	1.5	1.5	5.5	1.5	1.6	6.6	1.7	2.4
Grout Date	5/19'11	5/21'11	5/24'11	5/22'11	5/19'11	5/20'11	5/18'11	5/16'11	5/18'11	
Bbls. Cement	1	1.25	1.5	1.5	1.25	1	1.5	1.25	1.25	1.277

PROVING HOLES

Hole No.	866 *b*	867 *b*	868 *b*	869 *b*	870 *b*	871 *b*	Average
Test Date	6/5'11	6/5'11	6/8'11	6/8'11	6/12'11	6/12'11	
Gall's per Min.	0.9	0.9	1	1	1.3	2.3	1.2
Grout Date	8/31'11	9/5'11	8/31'11	8/31'11	8/30'11	8/28'11	
Bbls. Cement	1	1	1	1	1	1.25	1.04

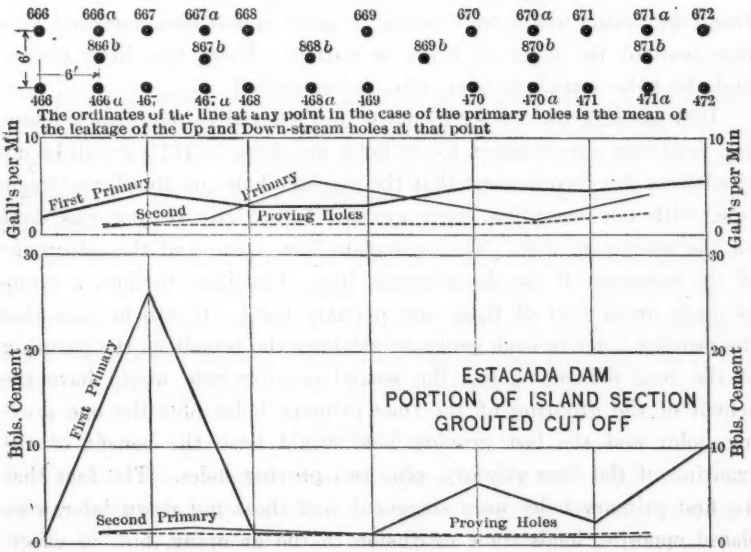


FIG. 8.

The same holes took the following quantities of cement:

	Barrels of Cement.		
	Maximum.	Minimum.	Average.
14 First primary.....	50.25	1	6.85
9 Second primary.....	1.50	1	1.27
6 Proving	1.25	1	1.05

Return to the Left Channel.

In the spring of 1911 several proving holes put down in the left channel and tested with the pump, as had been the original holes, notwithstanding the great quantity of cement that had been forced in, showed leakage and intercommunication between holes practically the same as for the holes put down before any grouting was done. When this was discovered, Mr. Fisher decided to put down new primary holes between those drilled the previous fall. These were tested every 10 ft. as drilled. These tests were so remarkable that the result of the work at this point will be discussed in detail.

By referring to the profile it will be seen that at depths varying from 12 to 40 ft. below the concrete, the normal conglomerate gave place to a water-bearing stratum. Through this the drills made good progress, but obtained a very small percentage of core. The washings from the drills, which were saved in great quantities, appeared as a fine sand of the color of brick or garnet. There was little caving, and the holes remained open after being drilled.

Disregarding the holes put down in 1910 and tested with a pump, and confining our attention to the holes put down in 1911, it will be noticed from the record sheet that the primary holes on the down-stream line, with one exception, were grouted before any testing was done on the up-stream line. The up-stream line, then, had the advantage of the grouting of the down-stream line. Consider, further, a group as made up of four of these new primary holes. It will be seen that the proving holes in each group would have the benefit of the grouting of the four primary holes; the second proving hole would have the benefit of the grouting of the four primary holes plus the one proving hole; and the last proving hole would have the benefit of the grouting of the four primary, plus two proving holes. The fact that the first primary holes were staggered, and those put down later were placed opposite, lends some confusion to the grouping, but, in effect,

still disregarding the holes put down in 1910, the drills were passed over the ground five times. This being the case, the holes drilled in each successive passage should show decreased leakage. That this progressive tightening did not follow and that the results were somewhat complicated and disappointing is shown graphically by the curves of Figs. 9 and 10. Tabulated, the averages appear as follows:

	Average Leakage of Holes, in Gallons per Minute, at Depths of:				
	10 ft.	20 ft.	30 ft.	40 ft.	50 ft.
Down-stream primary	10.3	15.0	29.9	33.2	47.3
Up-stream "	3.05	4.3	14.7	47.2	56.9
First proving	1.4	3.9	15.5	50.7	63.6
Second "	1.6	5.1	13.7	22.1	31.7
Final "	2.0	3.8	6.5	15.0	35.6

Cement was forced into these holes as follows:

	Totals.	Averages.
14 Down-stream primary	117.75 bbl.	8.38 bbl.
13 Up-stream "	79.25 "	6.09 "
11 First proving	21.50 "	1.95 "
12 Second "	20.25 "	1.84 "
17 Final "	26.50 "	1.55 "
67 Holes in all	264.75 "	3.95 "

If to the above are added the 28 holes put down in 1910, taking 172.5 bbl. of cement, and the 7 proving holes put down in the spring of 1911, tested with pump, and not in the above, taking 17.75 bbl. of cement, we have for this 84-ft. stretch of cut-off a total of 102 holes, aggregating 5 050 ft. of drilling, and into which was forced 455 bbl. of cement.

The average leakage of the final proving holes at each different depth, as shown by the foregoing table, expressed as percentages of the average leakage of the down-stream primary holes at the same depth appears as follows: At 10 ft., 20%; at 20 ft., 25.3%; at 30 ft., 21.7%; at 40 ft., 45.1%; at 50 ft., 75.2 per cent.

Back-Flow.

More strange, possibly, than the persistence with which the leakage continued, in spite of all the cement forced in, was the fact that with nearly all these holes, as with those put down in 1910, a very

DOWN-STREAM PRIMARY

Hole No.	637 *a*	638 *a*	639 *a*	640 *a*	641 *a*	642 *a*	643 *a*	644 *a*	645 *a*	646 *a*	647 *a*	648 *a*	649 *a*	650 *a*	Average
Test Date	9/8'11	9/29'11	9/15'11	8/26'11	8/22'11	8/29'11	9/1'11	8/19'11	8/30'11	8/24'11	8/30'11	8/23'11	8/29'11	8/20'11	
Gall's per Min	8	10	5	5	127	36.9	58.8	145	89.5	45.5	30	30	35.5	35.5	47.3
Grout Date	9/10'11	9/29'11	9/20'11	8/26'11	8/23'11	8/29'11	9/2'11	8/19'11	8/30'11	8/24'11	8/30'11	8/23'11	8/29'11	8/21'11	
Bbls. Cement	2	1.5	1.25	27	18.75	28	5	6.5	6.25	5	8.75	1.25	1	5	8.375

UP-STREAM PRIMARY

Hole No.	438 *a*	439 *a*	440 *a*	441 *a*	442 *a*	443 *a*	444 *a*	445 *a*	446 *a*	448 *a*	449 *a*	450 *a*	450 *b*	Average
Test Date	10/2'11	9/29'11	9/2'11	9/7'11	9/4'11	8/25'11	9/6'11	9/3'11	9/7'11	9/3'11	9/5'11	9/1'11	9/23'11	
Gall's per Min.	36.3	29	11.6	31	40.2	35.5	44.5	75	137	102	135	16.8	45.5	51.0
Grout Date	10/2'11	9/29'11	9/2'11	9/7'11	9/4'11	8/25'11	9/6'11	9/4'11	9/9'11	9/3'11	9/6'11	9/1'11	9/23'11	
Bbls. Cement	1.75	1.75	22	2.75	3	3	2.5	6.25	3.75	10	4.25	16.25	2	6.09

FIRST PROVING

Hole No.	839 *b*	840 *a*	841 *a*	842 *a*	843 *a*	843 *b*	844 *b*	845 *b*	847 *c*	848 *a*	850 *a'*	Average
Test Date	10/7'11	10/6'11	9/14'11	9/11'11	9/22'11	9/27'11	9/15'11	8/12'11	9/6'11	9/12'11	9/11'11	
Gall's per Min	4.7	41	26	45.5	135	10.9	89	135	59	68.7	121.5	67.0
Grout Date	10/7'11	10/6'11	9/20'11	9/11'11	9/23'11	9/27'11	9/20'11	9/13'11	9/19'11	9/13'11	9/11'11	
Bbls. Cement	1.5	1.75	2	2	2.5	1.75	2	1	2.5	2.5	2	1.95

SECOND PROVING

Hole No.	839 *a*	840 *b*	841 *b*	842 *b*	843 *c*	844 *c*	845 *c*	846 *a*	847 *b*	848 *a*	849 *a*	Average
Test Date	10/5'11	10/3'11	9/25'11	9/30'11	10/6'11	10/3'11	9/24'11	9/22'11	9/27'11	10/4'11	10/4'11	
Gall's per Min	27	39.1	25.4	20.1	33.5	29	15.6	78.5	18	30.5	32	31.7
Grout Date	10/5'11	10/3'11	9/25'11	9/30'11	10/6'11	10/3'11	9/24'11	9/22'11	9/27'11	10/4'11	10/4'11	
Bbls. Cement	1.5	1.75	1.75	2	1.75	1.75	1.5	1.5	1.75	2	3	1.84

FINAL PROVING

Hole No.	837 *b*	838 *c*	839 *c*	840 *c*	841 *c*	842 *a'*	842 *c*	843 *a'*	844 *a*
Test Date	10/11'11	10/13'11	10/12'11	10/12'11	10/11'11	10/13'11	10/11'11	10/13'11	10/10'11
Gall's per Min	5.5	8.5	22.3	37.5	33	45.5	46.5	30.7	60.5
Grout Date	10/13'11	10/15'11	10/15'11	10/15'11	10/15'11	10/15'11	10/15'11	10/16'11	10/15'11
Bbls. Cement	2	1.25	1.25	1.25	3	2.75	1.5	1	1.75

Hole No.	845 *a*	845 *b'*	846 *a'*	847 *a*	848 *b*	848 *b'*	849 *b*	850 *c*	Average
Test Date	10/10'11	10/14'11	10/10'11	10/10'11	10/11'11	10/15'11	10/11'11	10/11'11	
Gall's per Min	35.5	34	32.9	33	9.5	73	76	21.8	35.6
Grout Date	10/15'11	10/16'11	10/16'11	10/16'11	10/13'11	10/16'11	10/16'11	10/16'11	
Bbls. Cement	1	1.25	1.75	1.25	1.5	1.5	1.5	1	1.55

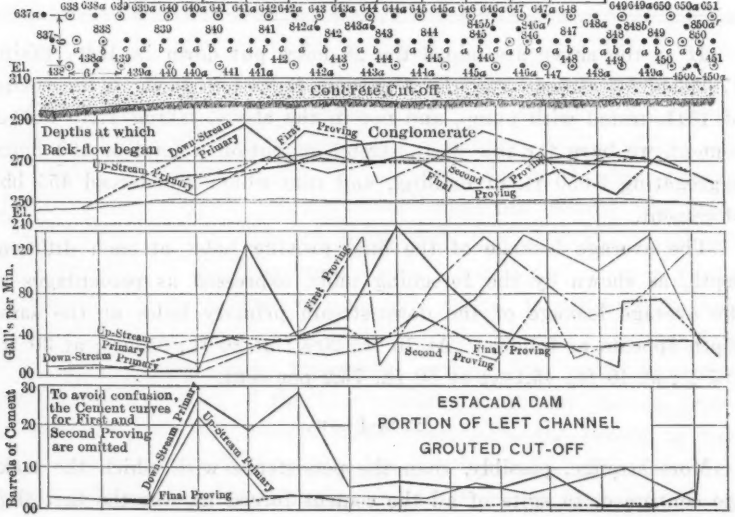
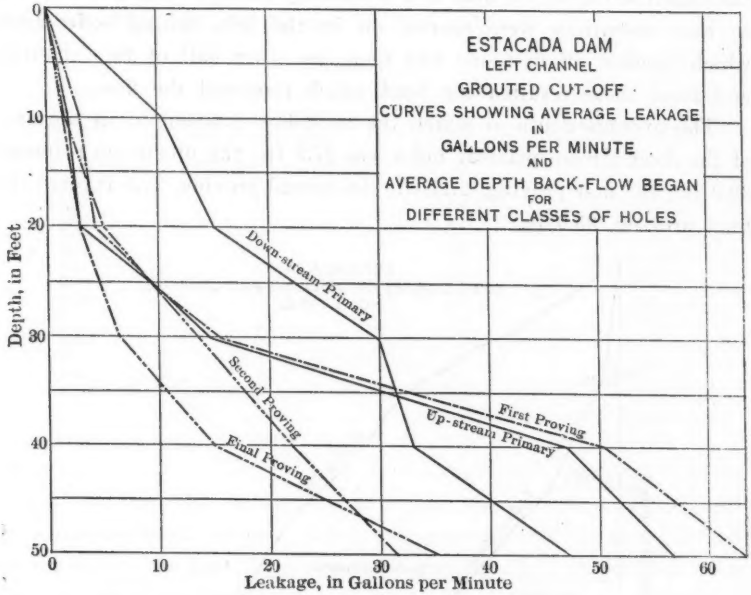


FIG. 9.


ESTACADA DAM
LEFT CHANNEL
GROUTED CUT-OFF
CURVES SHOWING AVERAGE LEAKAGE
IN
GALLONS PER MINUTE
AND
AVERAGE DEPTH BACK-FLOW BEGAN
FOR
DIFFERENT CLASSES OF HOLES
Down-stream Primary
Second Proving
Final Proving
First Proving
Up-stream Primary
Depth, in Feet
Leakage, in Gallons per Minute
0
10
20
30
40
50
60

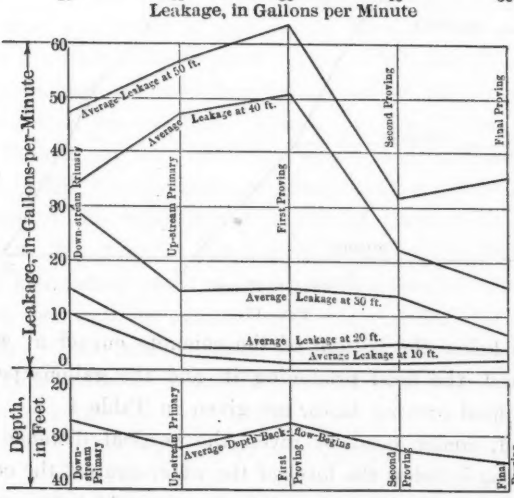

Leakage, in Gallons per Minute
Average Leakage at 50 ft.
Average Leakage at 40 ft.
Average Leakage at 30 ft.
Average Leakage at 20 ft.
Average Leakage at 10 ft.
Down-stream Primary
Up-stream Primary
First Proving
Second Proving
Final Proving
Depth, in Feet
Average Depth Back-flow Begins

FIG. 10.

insignificant head produced a back-flow from the river. At this time the construction of the dam had completely blocked the right channel, so that operations were carried on in the left, behind coffer-dams which blocked off first one and then the other half of this channel, and from these resulted the head which produced the flow.

The average depth at which the back-flow commenced in the case of the down-stream primary holes was 27.2 ft.; the up-stream primary, 32.0 ft.; the first proving, 27.7 ft.; the second proving, 32.5 ft., and the final proving, 35.0 ft.

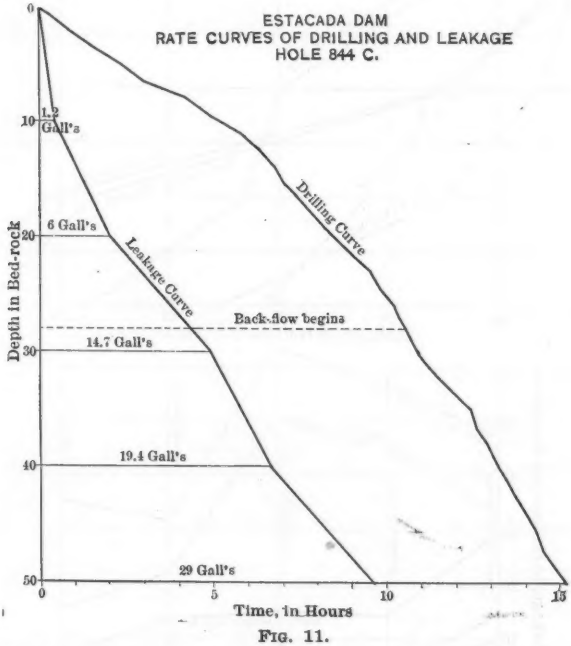


FIG. 11.

The depth below the bottom of the concrete cut-off at which this flow commenced, the head producing it, and the gallons per minute for 14 of the final proving holes, are given in Table 1.

In Table 1, under head, is given the vertical distance that the top of the casing is below the level of the water against the coffer-dam. This small head, averaging only 4.4 ft. for these 14 holes, produced a back-flow averaging 12.7 gal. per min., or one-third of the average leakage for the same holes when tested from a tank some 70 ft. above

the level of the water against the coffer-dam. In one hole, 845*a*, the back-flow, under a head of 7.7 ft., was 30 gal. per min., though the same hole, under test, leaked only 35.5 gal. per min.

TABLE 1.

Hole No.	Depth at which back-flow commenced.	Head.	Gallons per minute at full depth.
837*b*	..	....	0.0
838*c*	40	2.0	4.0
839*c*	35	5.6	13.0
840*c*	35	5.8	15.0
841*c*	32	2.9	11.0
842*c*	30	4.0	14.5
844*a*	30	6.7	17.0
845*a*	42	7.7	30.0
847*a*	33	7.2	22.0
848*b*	46	6.6	1.8
849*b*	28	1.0	8.0
850*c*	40	1.0	19.0
843*a*′	30	4.0	18.0
846*a*′	42	6.7	4.0
Average.....	33	4.4	12.7

Not only did the leakage continue and the back-flow persist, but the communication between holes was present even to the 17 final proving holes. In fact, by a system of piping, 13 of these final holes were connected and tested at one time. This, as well as other combination or multiple tests at other points of the cut-off, showed much less average leakage per hole than the average of the same holes tested singly. However, the fact that 17 holes put down along a stretch of cut-off 84 ft. long, in which 84 holes before them had taken 427.5 bbl. of cement, could communicate so freely that pressure from a tank on the crest of the dam applied to any of them would cause water to flow from the other 16 was poorly offset by the fact that 13 of these same holes when tested in combination showed an average leakage of 17.5 gal. per min. as against an average of 34.4 gal. when tested individually. This reduced leakage in the cases of the combination tests was interpreted as indicating that several holes intercepted the same seam or seams.

Grout in the Cores.

At one point, where a trench was excavated subsequent to the grouting, the cement could be traced as a white thread completely filling a seam ½ in. wide for 38 ft. This, of course, was only a few

feet beneath the surface. At greater depths the only evidence of the diffusion, aside from the decreased leakage in testing, would be the cement brought up as core or in the cement flakes in the wash from the drills. The grayish color of the wash, indicating that the drill was penetrating grout, was of much more frequent occurrence than the finding of solid grout in the cores. Of course, where the drilling of the proving holes followed in a few days or weeks the drilling of the primary holes, it was to be anticipated that the cement would be ground up by the attrition of the bits, the shot, and the harder particles of rock. After a lapse of several months, cement core might be obtained, but even then it would be best to use a double-tube core barrel. However, all cores were carefully inspected for any traces of cement, and, though some *bona fide* samples were found, they were very rare.

The words *"bona fide"* are used advisedly. At one time, owing to the poor showing being made by the grouting, there was serious thought of discontinuing it altogether. At this juncture, as the writer discovered later, at least one driller, moved by a commendable desire to see his job continue, joined short pieces of core with cement, which sample, deposited in the core box, was later seized on and carried up to the office as conclusive evidence of the diffusion of the grout.

Tests After Completion of Dam.

For a period of nearly 2 weeks after the pond had been allowed to fill, to admit of completing the excavation of the tail-race, the sluice-gates would be opened at night and closed during the day until such time as the water was nearing the crest. This, save for pools and pot-holes, left the left channel dry for several hours each day, and during these intervals careful daily inspections were made to detect any springs resulting from the 80 to 90-ft. depth of water above the dam. None was found, and the only seepage of any kind was very small in quantity, coming from well up on the left slope. The dam was completed at the time the fall rains set in, and this very insignificant seepage was believed to be due to the filling of the swales and depressions which occurred at intervals in the nearly level bench topping the left bank at the site of the dam.

When the water was flowing over the crest, a core drill was taken inside the dam and two holes put down. The first of these was next

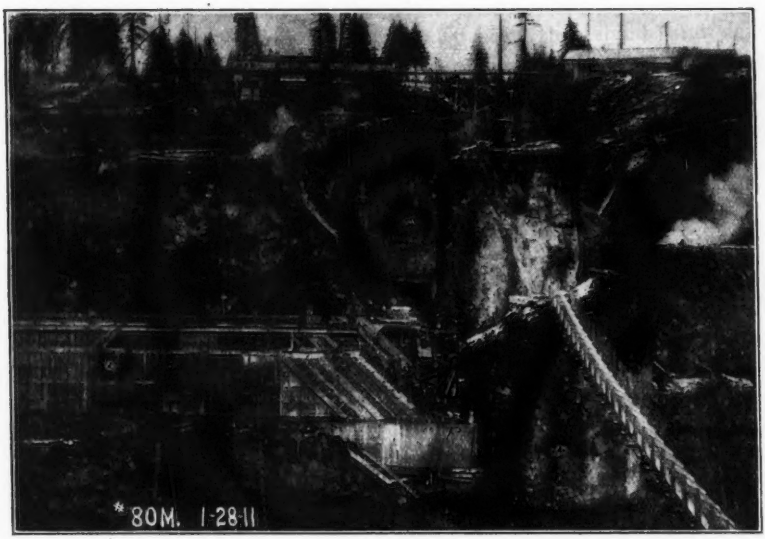


FIG. 12.—VIEW FROM LEFT BANK. ISLAND BEING CUT TO SLOPE OF DAM.

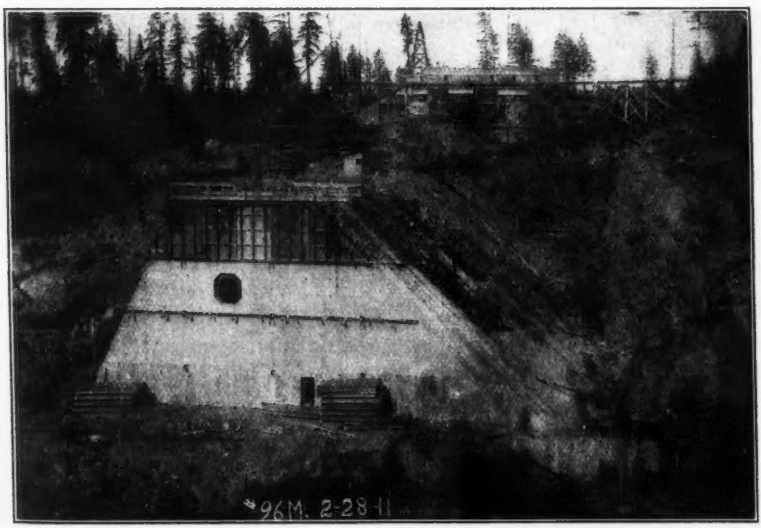


FIG. 13.—VIEW FROM LEFT BANK. DECK BEING EXTENDED TO INCORPORATE ISLAND IN DAM.

the left channel on what had originally been the island, and at a point 35 ft. down stream from the lower side of the grouted cut-off. Table 2, which is an abridgment of one appearing in Mr. Fisher's report to the owners of the plant, shows that water commenced to flow when the drill had penetrated the rock to a depth of 28.5 ft.

TABLE 2.—HOLE No. 1 000.

Elevation, top of casing.....328.2
" of bottom of rock at hole.....325.0
" bottom of concrete cut-off opposite hole..306
" grouted cut-off opposite hole.....248

Elevation, bottom of hole.	Elevation, water in pond.	Flow over top of casing, in gallons per minute.
296.5	375	First trace of flow.
289.5	376	0.9
280.5	387	2.2
270.5	387	5.4
260.5	387	5.4
249.0	387	6.6
245.5	387	7.5

The head, or difference of level, between the top of the casing and the water in the pond, with the exception of the first two determinations, is constant, and amounts to 59 ft.

A few days later, when the water in the pond stood at Elevation 388.4, the casing was extended upward with 1¼-in. pipe, great care being taken to have all joints tight. When the water finally came to rest it was found to be at Elevation 364.2 or 24.2 ft. lower than the water passing over the crest above.

The other hole was put down directly in the left channel, and opposite that point in which the leakage had been most serious and the rock most refractive to tightening by the grouting process. This hole, No. 1 001, was opposite Hole No. 644*a* and 30 ft. down stream from the lower line of the cut-off. The first trace of flow from this hole was at a depth of 22.9 ft. in the rock.

In this hole the casing was extended and the height to which the water would rise was determined at intervals as the drilling progressed.

In this case the difference between the top of the casing, the point at which the flow was measured, and the elevation of the pond was nearly constant, and amounted to 79.5 ft.

TABLE 3.—HOLE No. 1 001.

Elevation top of casing.....315.1
" of rock at hole.....308
" bottom of concrete cut-off opposite.....302
" " " grouted " " 248

Elevation, bottom of hole.	Elevation, water in pond.	Flow, in gallons per minute.	Elevation at which water came to rest.
285.1	.....	First trace of flow.	315.1
283.1	387.5	0.7	368.0
281.6	387.1	16.9	369.4
270.1	387.8	26.0	369.6
255.1	387.5	32.9	369.7
250.1	387.1	46.0	373.1
247.1	387.1	53.8	374.0

In this hole, when the bottom was at Elevation 282.6, or 25.4 ft. in bed-rock, the drill suddenly dropped 2 in. and the water came up in greatly increased force and volume.

TABLE 4.—COST DATA.

QUANTITIES:		
Total drilled, 555 holes		34 038 lin. ft.
Average per drill per 10-hour shift		13.2 " "
Average shot per drill per shift		1.1 lb.
Primary or outside holes, grouted	375 taking	1 526.50 bbl. cement.
Proving or middle holes, grouted	160 "	275.25 "
Tight holes filled	12 "	15.00 "
Holes lost	8 "	..... "
Setting casings, etc	.... "	125.25 "
Total	555 taking	1 942.00 bbl. cement.
Most cement in any one hole	.50 bbl.	
Average for 535 holes taking grout	3.37 bbl.	
COST:		
Labor, drilling	$19 842.60	$0.59 per lin. ft.
Labor, grouting	6 285.32	0.18 "
Cement, at $2.20 per bbl. at cut-off	4 272.40	0.12 "
Repairs, oil, waste, shot, etc	5 908.35	0.17 "
Depreciation on grouting plant, 50%	5 116.70	0.15 "
Total direct	$41 425.37	$1.21 per lin. ft.
Other charges were prorated as follows:		
General plant, camp, etc	$15 304.63	$0.45 per lin. ft.
Coffer-dams and pumping	5 121.63	0 15 "
Engineering and superintendence	6 414.91	0.19 "
Total cost	$68 266.54	$2.00 per lin. ft.

CONCLUSIONS.

In a piece of work of this character and magnitude, where much is experimental, nothing would be gained in attempting to present a

FIG. 14.—GENERAL VIEW, LOOKING DOWN STREAM. ISLAND CUT TO SLOPE OF DAM.

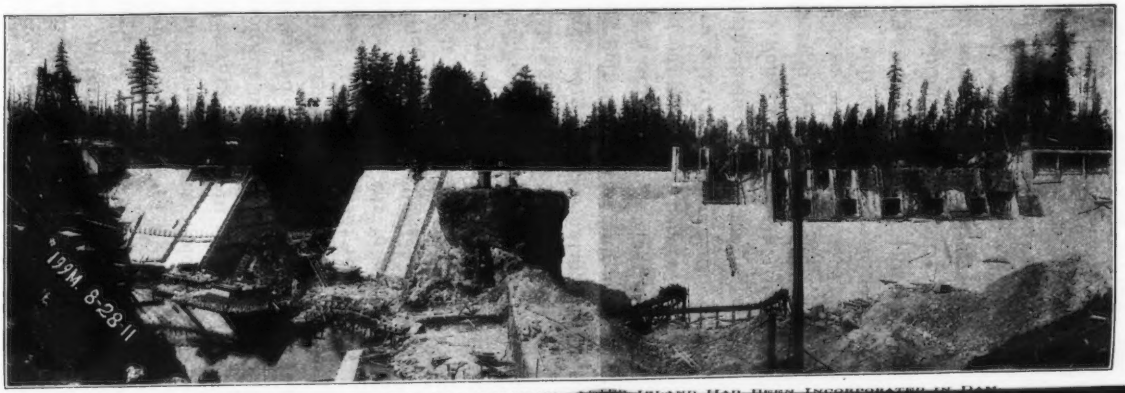


FIG. 15.—GENERAL VIEW, LOOKING DOWN STREAM. [illegible] HAS BEEN INCORPORATED IN DAM.

review or an epitome of the work as a whole, such, for example, as giving the average leakage of all the primary holes and contrasting this with the average for all the proving holes. Such a marshaling would impart, not information, but misinformation. One of the reasons for this is that any hole put down near one already grouted is in a sense a proving hole, whether in the middle or in an outside line of holes. Other reasons are: that the testing was done in part by pump and in part by gravity; in part from a tank at one elevation and in part from a tank at a different elevation; in part through casings set in the conglomerate before the cut-off trench was concreted and in part through casings set in the concrete with which the trench was filled. For these reasons, it seems best to rest the case with the consideration of the several portions of the work taken up in the preceding pages. These cover portions of the cut-off where the material was best as well as where it was poorest; where the grouting was most effective as well as where its benefits were least marked.

Several engineers from different parts of the country, who visited the work during the closing days of construction, attached considerable importance to the great flow from the two holes put down inside the dam, and to the fact that the pressure encountered was sufficient to raise the water to within a few feet of the crest of the dam. The opinion seemed to be that the pressure and flow proved the failure of the grouting process. To the writer, they mean merely that the water passing through the grouted cut-off at a depth of 25 to 30 ft. is stopped or greatly checked in its flow by the denseness of the rock in front, and is held down by the denseness of the rock above. The pressure and flow, in view of this, seem to indicate, rather, that the grouting was not needed, though this is assuming that the tightness of the surface rock is in no degree due to the grouting.

The writer is of the opinion that most engineers who have followed this description will have concluded, in so far as this job is concerned, that over a portion of the cut-off no grouting was needed, and that over those parts where it was needed it did little good. The initial tightness over a part of the right channel and island sections, as shown by the seepage test and pressure tests, and the great leakage, communication, and back-flow of the final holes in the left channel afford ample basis for such an opinion. However, calculating for a head of 80 ft. and assuming an efficiency of 50% for power delivered to

the customer, it is found that, at $33 per kw-year, a saving of 26 sec-ft. would pay 7% on the $41 000 representing the direct cost of the grouting. Whether or not this quantity of water has been saved, no one can say. Even had the first primary holes shown very free leakage and the final proving holes tested absolutely tight, it would not have been known, so different are the conditions when water is forced through a casing into a hole 50 ft. in the rock from those when the pond is filled and there is no such penetration to the interior rock.

In rock having continuous seams, the grouting process would probably be much more effective than in the "volcanic débris" of the Clackamas Canyon. In this, though the grout will fill the larger cavities and interstices, it cannot be called a success, in so far as its being an agent for providing means for actually changing the structural character of the formation, as was hoped when beginning the work.

The knowledge gained by the experience at Estacada may be summed up as follows:

(1) Do all drilling, testing, and grouting through casings set in the concrete cut-off.

(2) Do all testing from elevated tanks, and not by pump.

(3) Test and grout each hole as soon as drilled, and for a few days thereafter keep the drills away from the probable zone of diffusion.

(4) In grouting, especially at high pressures, it is best to close the valve before the tank is entirely empty, as the air following the grout into the hole is apt to make trouble.

(5) Begin with a comparatively thin grouting mixture and, if taken freely, thicken until each succeeding batch requires either an increased time for discharging or an increased pressure. To force charge after charge of thin grout into a hole probably means in a great measure the wasting of cement.

The writer's opinion, now, is that either a single row of holes with close spacing or two rows of holes very close together in an up-and-down-stream direction, with casings staggered, is preferable to the triple line used at Estacada. At the Lahontan Dam, of the Truckee-Carson Project,* two rows of casings were put down, the distance be-

Engineering Record, March 29th, 1913, p. 340; and *Engineering News*, April 3d, 1913, p. 647.

tween the rows was only 2 ft., and the distance between the holes of each row, 3 ft. As the casings in the two rows were staggered, it virtually amounted to a casing every 18 in. Over one section of this work, every fourth hole in the up-stream line of casings was first drilled, tested, and grouted. When the end of the cut-off was reached, the drills were returned to the beginning and the middle hole in each space was drilled, tested, and grouted, after which the drills were returned a third and a fourth time, drilling the remaining holes. This made every hole after the first, a proving hole, and as practical tightness was secured with the drilling of the up-stream line, the casings of the down-stream line were not needed, and were drilled through in only four or five instances as final proving holes.

The experience at Estacada indicates that the grouting should in no case be relied on in lieu of the usual concrete cut-off. There are two reasons for this:

(1) The efficiency of grout as a curtain-wall cannot be foretold.
(2) The proper diffusion of the grout can be secured only when the concrete of the cut-off closes the surface seams and confines the pressure to a depth at which it may be effective in tightening the underlying material.

Where very porous material is encountered below the practicable limit of depth for a concrete cut-off which, as in the case of the left channel at Estacada, proves refractive to the tightening by the grouting of isolated holes, the desired end might be secured by drilling a great number of holes close together and by springing the rock with small charges of powder from the bottom up to the top of the water-bearing stratum, thus shattering and loosening up the intervening material so that the grout would then form an impervious barrier.

As already stated, the pressure testing, though affording one means of measuring the effect of the grout, by no means proves it to be necessary.

In the case of a storage dam, the best plan would seem to be to set the casings in an offset cut-off just up-stream from the heel of the dam, as was done along part of the dam at Estacada. Through these casings a hole could be put down every 12 to 16 ft. to make sure that there were no large crevices. These casings should be tested and grouted and the need for further drilling and grouting left to be determined

by the filling of the pond. If the leakage were excessive and the seams did not tighten by the natural silting processes, the pond, at time of minimum storage, could be entirely emptied and the grouting continued. In this case it would be necessary to grout only at those points where seepage showed it to be needed, and, if tightness were secured, the engineer would have absolute proof of the efficiency of the process. Of course, with an earth dam like that at Lahontan, already mentioned, this method is impossible, and grouting must precede the construction of the dam proper.

In the case of a power dam, the same plan might be followed, and drilling and grouting done by drawing off the water at the time of minimum flow. Whether to do this or to carry the grouting along with the building of the dam, as was done at Estacada, is a problem the solution of which depends on what the minimum flow power is worth and whether the load might, without any great inconvenience, be transferred to other plants for a short time.

The consulting engineers on this work were Messrs. Sellers and Rippey, of Philadelphia. They were represented at various times and in various capacities on this and other projects of the Company, including the proposed 150-ft. dam on the Clackamas River, by Messrs. William H. Cushman and H. V. Schreiber, Members, Am. Soc. C. E., and Messrs. W. L. Fitzgerald and S. C. Hulse, Associate Members, Am. Soc. C. E.

A. Gardner, Assoc. M. Am. Soc. C. E., represented the Ambursen Hydraulic Construction Company, of Boston, owners of the hollow-dam patents under which the dam was built, and L. I. Fletcher, Assoc. Am. Soc. C. E., was Superintendent for the contractors, the Puget Sound Bridge and Dredging Company.

Mr. T. W. Sullivan is Chief Hydraulic Engineer of the Portland Railway, Light and Power Company, owners of the plant, and Mr. F. R. Fisher was the Resident Engineer on the dam. The writer was Assistant Engineer of the same Company, and was connected with the work from the time of making the survey to the delivery of power.

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PAPERS AND DISCUSSIONS

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THE DIVERSION OF IRRIGATING WATER FROM ARIZONA STREAMS.

By A. L. Harris, Assoc. M. Am. Soc. C. E.

To be Presented February 18th, 1914.

Most of the irrigating water obtained from Arizona streams comes from the higher mountains. Very little rain falls in the lower and more level country, where cultivation is practicable. For ages the typical stream has gathered its load of silt, sand, and gravel in these mountains and forced it, by reason of strong slopes, through the canyons and down to the low country, where grades become less, and it is spread out evenly in the broad, gently sloping, cultivable valleys. Usually, such valleys show evidences of having been flowed by slack-water for periods long enough to deposit deep surface beds of fine silt. The overflow has eventually cut its way out through surrounding hills, and worn down the outlet below the valley level. A channel has then been cut across the valley floor in the alluvium, toward which channel the surface slopes and drains on each side. The irrigated valley is now like a great shallow dish, inclined a little, and with the bed of the stream running across it in the direction of its slope. By intercepting the river water with a dam where it enters the valley from the surrounding hills, it may often be diverted into two canals, one to the right and one to the left of the stream. Throughout a wide

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

circuit around the edges of the dished valley the canals are located, with gentle grades to insure the flow and with the water surface generally a few inches above the natural ground to enable the water to be taken out at intervals. Of course, it is then possible to draw water into ditches from one of these canals to irrigate any piece of land between it and the river, as there is always a slope in that direction. As the river seldom flows through the middle of the valley, the area of irrigable land under each main canal is not the same. Indeed, it often happens that practically all the irrigable land of a valley lies on one side of the river. The great Salt River Valley about Phœnix has about twice as much irrigable land on its north side as is found on the south, and the Agua Fria and Buckeye Districts, on the Agua Fria and Gila Rivers, respectively, each utilize practically only one side of the stream.

Were it not that the arid nature of the country still allows erosion phenomena to go on vigorously, the provision of works for diverting the irrigating water would be much simpler. For some years the writer has frequently been called on to design diversion works in connection with both United States Government and private irrigation projects in this region, and this paper deals with the principal features and conditions relative to this experience.

Conditions to be Met.

The natural conditions in the locality of diversion present a stream having a flow which varies between very wide limits, and subject to very sudden changes. The floods often carry large quantities of water in great rushes, bearing down with them all kinds of drift—on the surface, in suspension, and grinding along the bottom—wood, silt, sand, gravel, and boulders. The works will be subject to much heat and dryness, but no ice (except on the plateaus of northern Arizona). The range of surface temperature is 160° or more. It is difficult to find an all rock foundation, as the river bed generally consists of a deep canyon, with rocky sides, which has been nearly filled with drift.

For the requirements of irrigation, diversion works must be such that the flow of water shall not be interrupted during long periods in the growing season, which, by the way, in southern Arizona, is nearly 12 months in the year. Canals should be kept free from sand and gravel, but the fine silt carried in suspension is valued highly by the

farmer, for keeping up the strength of the soil, very little other fertilization being necessary. If the system includes a storage reservoir somewhere up stream, much of this valuable silt, unfortunately, is deposited in the bottom of the reservoir, where, for the most part, it is not only useless but in time becomes a serious problem. The capacity to divert and distribute promptly unusually large quantities of water in flood times is generally required. Surface drift must be kept out of the canals. The layout of the whole works must also be such that the banks of the canals, where near the river, will be protected from the washing of the waste waters when floods pass down the river.

The Diverting Dam.

For diverting water into the canals, some kind of dam is generally required, although in special cases a subterranean collector conduit is used. It is generally necessary to raise the elevation of the water considerably in order to place the canals above the reach of flood water and to raise the surface level in them as much as possible for serving the greatest area of irrigable land.

The original diverting dam for irrigation was merely an obstruction of brush, stone, and earth projecting into the stream (as a wing) or across it, and deflecting the water into a ditch at about the natural level of the stream bed. Any small rise in the river destroyed it, and it had to be rebuilt. The point where the river water entered the canal was generally chosen at a place naturally suited to withstand the wear of flood, as by cutting through a projecting rock ledge or boulder point. The most serious expense and loss with such a canal head was not that of rebuilding the brush and stone dam, but the loss of crops due to the interruption of irrigation. However, by the hard and persistent work of the pioneers, most of the irrigated districts in Arizona were started and developed from such a beginning.

The next step in improvement was by the use of timber and rock-filled crib dams. Although a dam of this type can doubtless be made to perform the required service, its common history in Arizona has been that, because of insufficient protection of the foundations, or by ill chosen dimensions governing discharge, it has been sooner or later breached by a flood, causing heavy losses. In the absence of accurate flood data, and even when such data are at hand, it is difficult for the designer to appreciate the destructive action to be provided against

in damming these streams, except after first-hand experience. Usually, the river presents a very shrunken appearance, because of dry weather, and the intervals between extreme floods, often extending through a considerable number of years, give time to cover up and age the evidences of their action.

In this climate, timber is an even more temporary material than usual. All its possibilities for warping and checking are brought out by wetting followed by much heat and dryness. The necessary iron fastenings are hastened in rusting by alkalies often contained in the water. As this timber must usually come from Oregon and Washington, by way of Southern California, it is also expensive. At the present and prospective prices of cement, the expense of timber-crib dams in this locality is not justified by their enduring qualities.

Some diversion dams have been built of cemented stone masonry, but of late the most suitable and available material has proved to be concrete. Concrete materials are found at or near almost any diversion site. The excavated materials from the foundation trenches often prove to be quite suitable. Thus it appears that, for most cases in this locality, a concrete dam and intake works suitably designed and constructed is, both economically and structurally, about the ideal structure for diverting irrigating water. The concrete dam is usually carried entirely across the stream, but, on account of expense and poor foundation conditions, it sometimes appears advisable to build, next to the intake gates, a wing-dam of permanent construction, which acts as a spillway for moderate floods; and to maintain a dike of earth with a higher crest to a closure on the opposite river bank. This dike is expected to go out in the heavier floods, and then must be replaced.

The main features of the complete diverting dam, which is practically always of the overflow type, are the foundation, the apron, the rollway, the sand sluices, and the canal intakes.

Where bed-rock cannot be secured for foundations, excavation will generally discover beds of heavy boulders deposited by the stream, tightly packed together and chinked in with gravel and sand. On account of the usual position of the diverting dam at the mouth of a canyon, as mentioned previously, the heavier products of erosion coming from the mountains can be confidently looked for in the bed of the river. The deposits grow progressively coarser as the depth increases. A series of rather heavy walls is laid out and built in

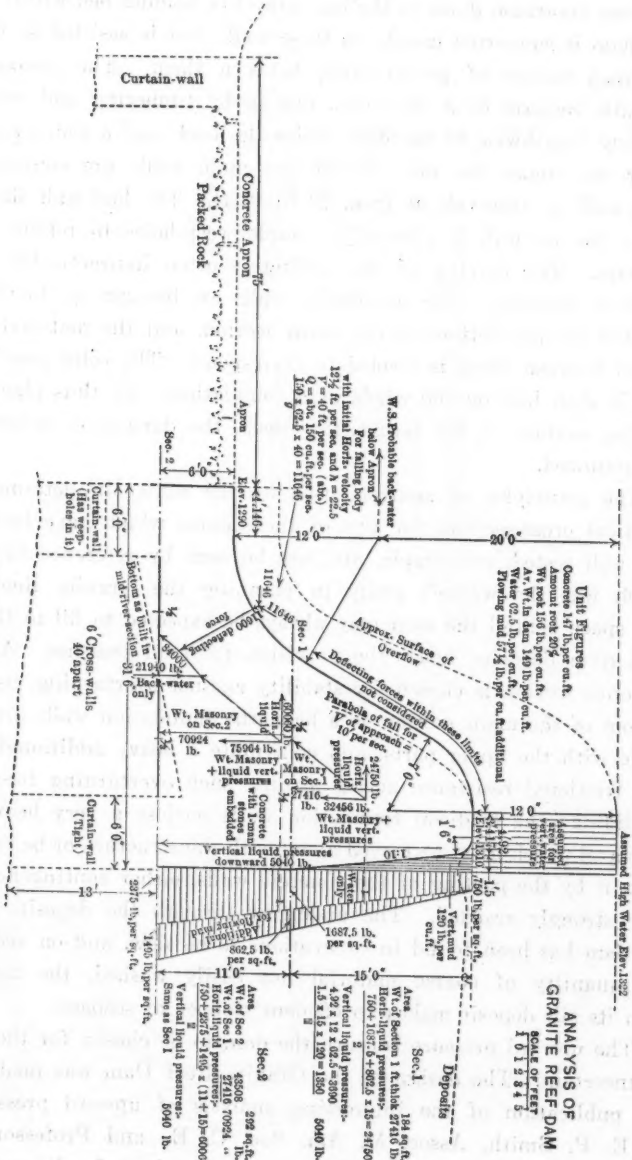
Curtain-wall
Concrete Apron
Packed Rock
Apron
Elev. 1250
Sec. 2
6'0"
4'146"
W.S. Probable back-water below Apron for falling body with initial Horiz. velocity 12½ ft. per sec. and D = 40 ft. per sec. / (abt.) Q = abt. 150 cu. ft. per sec. 1.0 x 62.5 x 40 = 11646
11646
11646
Sec. 1
12'0"
20'0"
Unit Figures
Concrete 147 lb. per cu. ft.
Amount rock 20%
Wt rock 156 lb. per cu. ft.
Av. Wt. in dam 149 lb. per cu. ft.
Water 62.5 lb. per cu. ft.
Floating mud 97½ lb. per cu. ft. additional
Approx. Surface of Overflow
Deflecting forces within these limits not considered
Parabola of fall for V of approach = 10 per sec.
1'0"
Assumed zero for Horiz. pressure
Elev. 1230
Assumed High Water Elev. 1232
12'0"
1'6"
1.10
1.5
760 lb. per sq. ft.
Water only
Vert. mud 120 lb. per cu. ft.
1687.5 lb. per sq. ft.
862.5 lb. per sq. ft.
2875 lb. per sq. ft.
1495 lb. per sq. ft.
11'0"
15'0"
2'
Sec. 2
Area 48508 sq. ft.
Wt. of Sec 1 27416 "
Horiz. liquid pressures- 70924 "
730+2875+1495 (11+15=60000 lb.
Vertical liquid pressures- 6040 lb.
Same as Sec 1
Sec. 1
Area 184 sq. ft.
Wt. of Section 1 ft. thick 27416 lb.
Horiz. liquid pressures- 21750 lb.
760+1687.5+862.5 x 15=21750 lb.
Vertical liquid pressures- 6040 lb.
4.92 x 12 x 102 = 6040
1.5 x 15 x 150 = 1350
Deposits
ANALYSIS OF GRANITE REEF DAM
SCALE OF FEET
0 1 2 3 4 5
Curtain-wall (Right)
Curtain-wall (Left) holes in it
6'0"
6'0"
Bottom as built in mid-river section
21940 lb.
Back-water only
6 Cross-walls 40' apart
Wt. Masonry on Sec. 2
70924 lb.
75904 lb. Wt. Masonry + liquid vert. pressures
Horiz. liquid pressures
60000 lb.
Wt. Masonry on Sec. 1
27416 lb.
24750 lb.
32456 lb. Wt. Masonry + liquid vert. pressures
Concrete with 3-cornered stones embedded
Vertical liquid pressures downward 5040 lb.
13'

FIG. 1.

trenches excavated down to the best gravel or boulder bed within reach. The dam is supported mostly on these walls, but is assisted by the undisturbed masses of gravel lying between them. The arrangement of walls consists of a deep one, put in by timbering and pumping, running lengthwise of the dam under the heel, and a somewhat shallower one under the toe. These two main walls are connected by cross-walls at intervals of from 20 to 40 ft. The heel-wall should be tight; the toe-wall is pierced by ample weep-holes to relieve underpressure. The driving of sheet-piling is often impracticable on account of boulders. The foundation walls are brought up to the level selected for the bottom of the main section, and the material in the spaces between them is leveled to correspond. The solid gravity section is then laid on the whole as a foundation. By thus placing the bearing surface of the foundations deep, the danger of undercutting is minimized.

The principles of analysis used by the writer in determining a practical cross-section, for two of these dams which have been built and well tested, are simple, and can be seen by reference to Fig. 1, which was the writer's study in planning the Granite Reef Dam. The space behind the dam may always be expected to fill to the crest with river deposits, hence the provision for mud pressure. Although the cross-section is chosen for stability against overturning, using the bottom of the main section as a base, the foundation walls are monolithic with the upper parts, and contribute a heavy additional weight and frictional resistance acting against such overturning forces. As to sliding: the gridiron foundation walls enclose a very heavy mass of gravel which must be moved along with the structure or be separated from it by the parting of the concrete walls, either contingency being very strongly resisted. The underflow through the deposits beneath the dam has been found in excavations to be slow, and on account of the quantity of coarse material not easily washed, the first flood with its silt deposit makes an efficient stopper of seepage.

The upward pressure beneath the dam to be chosen for these cases is uncertain. The design of the Granite Reef Dam was made before the publication of the interesting analysis of upward pressures by G. E. P. Smith, Assoc. M. Am. Soc. C. E., and Professor H. C. Wolff, of the University of Wisconsin, contained in discussions on "Dams on Sand Foundations," by Arnold C. Koenig, Assoc. M. Am.

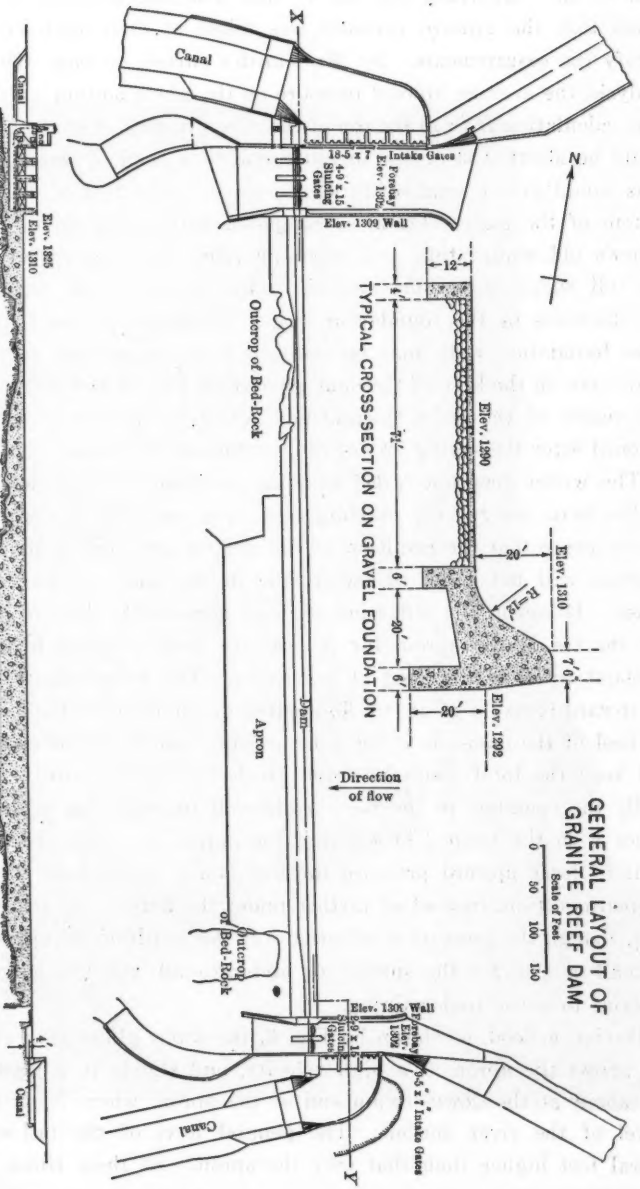
GENERAL LAYOUT OF GRANITE REEF DAM
Scale of Feet
0 50 100 150
TYPICAL CROSS-SECTION ON GRAVEL FOUNDATION
Canal
Intake Gates
Forebay
Elev. 1302
Sluicing Gates
Elev. 1309 Wall
Outcrop of Bed-Rock
Elev. 1290
Elev. 1310
Elev. 1292
Dam
Apron
Direction of flow
Gravel
Elev. 1825
Elev. 1310
N
X
Y

FIG. 2.

Soc. C. E.* Applying the test of that analysis, however, to Fig. 1, shows that the upward pressure was taken at more than enough to satisfy the requirements. By Mr. Smith's curves, on page 199 of that analysis, the average upward pressure on the whole bottom width of the dam, calculating H from the top of the apron instead of at the river bed, would be about 0.33 of the pressure due to a head of water of 20 ft. This would give a total of 18 000 lb. per running foot of dam on the bottom of the main section, as compared with 21 940 lb. used in the writer's old study; this, too, when applying the average pressure to the full width of bottom without excluding the 12 ft. taken up in the thickness of the foundation walls. Pressures on the bottoms of these foundation walls may be assumed to be greater per square foot than those on the base of the dam where they join it, but will not equal the weight of the walls themselves. All these upward pressures are lessened after the silting up has occurred above the dam.

The writer does not forget that the analysis, the test of which he applies here, assigns the maximum under-pressure at low water. He is also aware that the resultant of the upward pressure of Mr. Smith's diagram will not come in the middle of the base, as taken by the writer. However, the difference of head between the heel of the dam and the toe of the apron, for the case of flood assumed herein, will be about equal to the head at low water. The writer also finds that an upward pressure of 18 000 lb. applied to one-third of the base from the heel of the dam—as if the pressure diagram were a triangle—will still keep the total resultant force inside the middle third.

By the openings in the toe curtain-wall for relieving pressure, together with the method of jointing the apron, it is believed that the point of least upward pressure for the dam is at or near the toe of the main section, instead of farther under the apron. As will be seen later, though the pressure is relieved over the width of the apron, there is small chance for the surface or underground water to carry away material to cause undermining.

During a flood, as shown by Fig. 3, the water glides over the ogee and across the apron at a high velocity, and stands in a great wave just about at the down-stream end of the apron, where it strikes the gravel of the river bottom. The general level of the tail-water is several feet higher than that over the apron. At these times, so far

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 193.

FIG. 3.—SALT RIVER PROJECT: GRANITE REEF DIVERSION DAM IN FLOOD.

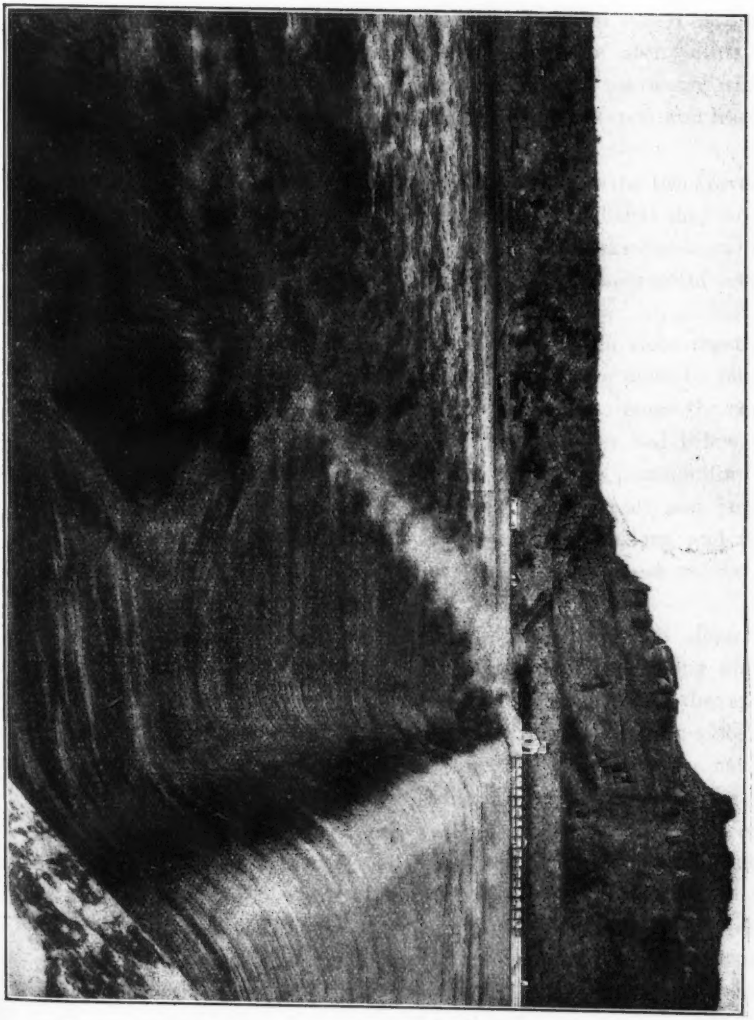

FIG. 3.—SALT RIVER PROJECT: GRANITE REEF DIVERSION DAM IN FLOOD.

as the hydrostatic pressures under the structure are concerned, the apron with its open joints must be an area where under-pressures from above the dam and from the tail-water below are both relieved. It can be readily seen that with a tight apron of small thickness, dangerous upward pressures might be brought to bear on it. It is quite conceivable that the swiftly passing water over the open joints of the apron exerts some suction or injector action on the water which rises from beneath, thus assisting the escape of those waters and lessening their pressures.

No expansion or settlement joints were provided in the two Government dams referred to, and no evidences have appeared that they would be desirable. The dams have developed a few transverse cracks, which are no detriment, as they will silt up. There is also no sign of settlement shown by the cracks.

It is well to keep the dam and the various gates close together, usually in one continuous structure. The sand sluices must be placed with the object of disposing of sand which threatens to enter the canal intake. This sand can best be deposited in the river bed below the dam, to which place it is usually sluiced through a passage for the purpose. A good arrangement is to combine the gates and intake structures with the abutment wall at the end of the dam, and it is very desirable for this purpose to have bed-rock at these points for foundations.

Very careful attention must be paid to the apron laid along the bottom of the stream to protect the toe of the dam from being undermined. Even where a bed-rock foundation is practicable, the apron has been found to be a part of the structure very vital to its stability. Its surface should be placed at a level from 2 to 4 ft. below the natural bed of the stream. A gravel bar will then form just beyond its downstream edge which will back the water over the apron and keep a water cushion always at the toe of the dam for receiving the shock of the overflow. The width of apron sufficient to protect the toe of the dam varies according to the height and volume of the overflow and the consequent scouring energy of the water. The depth of high water above the crest will also influence the width. The apron is of concrete, but must not be connected to the dam, or in one continuous sheet. Being laid on beds of gravel and boulders near the surface, it will heave and settle more or less, and, if laid in a continuous sheet,

this action cracks it into irregular, sharp-angled slabs which are likely to be shifted and undermined by the flowing water.

A satisfactory apron was made for the Granite Reef Diversion Dam by preparing a bed of packed clean boulders in which many of the stones projected prominently. A bed of concrete, about 18 in. thick and divided into 10-ft. square slabs by vertical joints, was laid on top of this foundation. On account of the adhering boulders on the under sides of the slabs, together with their weight, they are adapted to offer great resistance to sliding. The dam raises the water level 20 ft. The apron was 75 ft. wide, down stream from the toe to the dam. Besides dividing the apron into heavy slabs of definite shape, the joints between the slabs also served to relieve the upward pressure of underflowing water during the first few months, while the river bed above was sealing itself up with silt and sand. A curtain-wall, braced with piles and waling pieces, was used to strengthen the unsupported edge of the apron. A similar apron has proved satisfactory at the Diversion Dam of the Government Power Canal on Salt River, 19 miles above the Roosevelt Storage Dam.

The rollway has a surface sufficiently outside the path of free fall to be sure that an adhering nappe is preserved. The writer does not favor a turned-up toe for the main section, as it tends to cause increased agitation of the water on the main structure. It is better to let this energy be spent on the apron and on the bar below it. This result was secured at Granite Reef, as illustrated by Fig. 3.

Gates.

The entrance of sand into the canals has been resisted by using the principle of skimming the less turbid water from near the surface. A forebay is walled off on two sides, in the form of a comparatively deep channel running directly in front of the row of intake gates; it is open at the upper end and given outlet at the lower end by large sluice-gates emptying through an opening in the dam. The intake gates draw water from this forebay at a level considerably above its bottom, and the sluice-gates, which have large capacity, draw from the very bottom of the forebay, which has a good slope toward them. In entering the forebay, the motion of the water is checked by the increased cross-section, and the heavier sand grains settle to the bottom. At intervals the sluice-gates are opened suddenly by a hydraulic

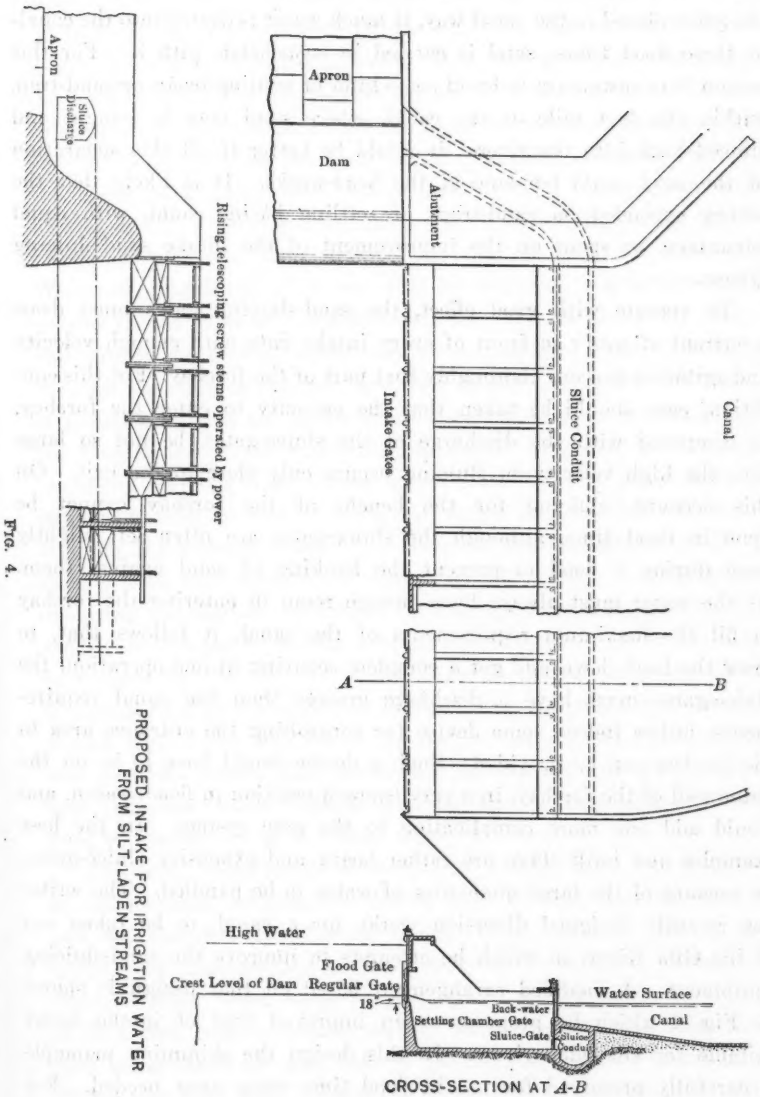

FIG. 4.

PROPOSED INTAKE FOR IRRIGATION WATER FROM SILT-LADEN STREAMS

piston, and a powerful rush of water washes out the sand. During a flood the water eddies about, and less sand is allowed to settle. With the gates placed in the usual way, if much water is drawn into the canals at these flood times, sand is carried in suspension with it. For this reason it is customary to build some kind of settling basin or sand-trap, within the first mile of the canal, where sand may be caught and sluiced back into the river. It would be better if all this separation of the sand could be done at the head-works. It is likely that the money expended on sand-traps or settling basins could, with equal advantage, be spent on the improvement of the intake sand-sluicing system.

To operate with good effect, the sand-sluicing gates must draw a current of water in front of every intake gate with enough velocity and agitation to scour thoroughly that part of the forebay. For this condition, care should be taken that the capacity to enter the forebay, as compared with the discharge of the sluice-gates, be not so large that the high velocity in sluicing occurs only close to the exit. On this account, sluicing for the benefit of the forebay cannot be done in flood time, although the sluice-gates are often left slightly open during a flood to prevent the banking of sand against them. As the water must always have enough room in entering the forebay to fill the maximum requirements of the canal, it follows that, to draw the level down and get a complete scouring at one operation, the sluice-gates must have a discharge greater than the canal requirements, unless indeed some device for controlling the entrance area to the forebay can be provided. Such a device would have to be on the outer wall of the forebay, in a very exposed position in flood season, and would add one more complication to the gate system. In the best examples now built there are rather heavy and expensive sluice-gates, on account of the large quantities of water to be handled. The writer has recently designed diversion works for a canal, to be taken out of the Gila River, in which he attempts to improve the sand-sluicing equipment. A modified arrangement based on that design is shown by Fig. 4 which he proposes as an improved type of intake works suitable for silt-laden rivers. In this design the skimming principle is carefully preserved for use in flood time when most needed. For the ordinary low-water conditions, the water is clearer, and is taken from the river over a series of wide, shallow crests, which reduce the

slope of the river bottom in the approaches to one so gentle that coarse materials will hardly be carried close to the gate entrances.

In flood times the forebay of the old type fills with gravel which is not easily sluiced out afterward. With the proposed arrangement, the lower part of the intake gate, called the "Regular Gate" is closed as soon as the flood rises 2 or 3 ft. above the crest of dam, and water for the canal is taken over the top of this gate, being controlled by the upper part, called the "Flood Gate", which closes down on the first as a sill. Gravel rolling along the river bottom during the flood passes on over the dam finding no place to enter or accumulate. Whatever gravel and sand does find its way into the intake gates is caught in the settling chambers (one for each intake gate) where it can always be sluiced under uniform conditions from the bottom, through the sluicing conduit, and back to the river below the dam. It is planned to have the upper leaf of the inside gates rise and cut off the back-water of the canal simultaneously with the opening of the lower, or sluicing leaf. These gates, however, can be worked independently, and, in case of need, the water may be shut off from the sluicing chambers entirely. It is believed that an intake of this type can be built and operated as cheaply as the other. A suitable power operating mechanism, which can be uncoupled for hand operation, can readily be designed for the proposed arrangement, as it has already been done for the old. It is also believed that sluicing the collected materials from one chamber of moderate size at a time, where a shallow stream with sharp fall can be secured to agitate and cut the deposit, will be more certain and thorough. All the sluicing gates discharge into the same waste conduit, as only one or two settling chambers need be flushed out at the same time.

The most satisfactory sluicing and intake gates yet devised are of the common rectangular sliding type, of cast iron or sheet steel, and operated by rising screw stems. They move on bronze sliding strips, and close on oak or metal sills at the bottom. Those at the Granite Reef Dam, built by the U. S. Reclamation Service,* are of thin cast iron, of arched section on the compression side, the tension being taken by steel rods at the back and placed across the bow of the cast-iron shell. They are light, strong, and durable. In this case, the sluice-gates are operated by hydraulic pressure pumps, the

* *Engineering News*, January 7th, 1909.

transmission being by heavy chains running over sheaves; they are weighted with concrete for closing by gravity. At the Granite Reef Dam, a very excellent feature of all the gates, which were designed by F. Teichman, M. Am. Soc. C. E., is that, on the pressure side, where sand, etc., is likely to bank against them, they present smooth fronts, instead of a system of deep ribs, as often built in the past.

In designing gates for these situations, the question of the proper coefficient of starting friction to use for the bronze sliding ways comes up. The writer's experience has shown that this coefficient is much larger than published authorities known to him would indicate. In order to make a reliable determination of the coefficient under actual working conditions, the writer disconnected the raising mechanism of a 5 by 7-ft. cast-iron gate, having machine-bronze sliding ways, in the power canal at Roosevelt, and in its place attached a long timber lever resting on a fulcrum made of a piece of round steel shafting held between flat steel plates. A platform for carrying weights was then suspended from a definite point on the lever arm, and the apparatus was used to weigh the starting resistance. The actual weight of this particular gate was known from the inspector's weight obtained at the time of its receipt from the manufacturer. The timber lever was weighed and its weight per linear foot assumed to be constant. A correction for the weights of the lever arms was then made.

Statement of Conditions of Test.—

Water pressure on only one side of gate;	
Size of gate opening	5 by 7 ft.
Area subjected to water pressure on the gate, measured on the center line of closing strips.	38.7 sq. ft.
Depth of water at sill of gate	8 ft. 8 in.
Head on center of pressure area	6 ft. 0 in.
Total water pressure on gate	14 500 lb.
Weight of gate	4 300 lb.
Sliding strips	Machined bronze.

Two experiments were made, as follows:

First Experiment.—To determine coefficient of starting friction after the gate had been closed tight for several weeks:

Result:	Frictional resistance	9 062 lb.
	Coefficient of starting friction	0.625

Second Experiment.—To determine coefficient of starting friction with the gate raised off the sill about ½ in. and water escaping under its lower edge:

Result: Frictional resistance..... 8 985 lb.
Coefficient of starting friction..... 0.62

The turbid condition of the water, perhaps, is the chief cause of the increased size of the coefficient.

In nearly all cases it would probably pay to use gasoline engine power for operating the gates. These engines are now so common that one suitable for the power required may always be found, and speed in opening and closing is essential. They should be designed in such a way, however, that hand power may be applied in case of necessity.

Trash-racks and booms, for protecting the gates from driftwood, have not been found necessary. Most of the driftwood comes down in flood time, and then it is nearly always carried in the middle of the stream and over the dam, as it collects in the stronger current in that part of the river. If the dam is placed below, or on, a sharp bend in the stream, which is not good policy, the drift, of course, will be thrown close to the outside shore and will need to be guarded against.

Special Cases.

In Arizona there is much more land fit for irrigation than can ever be properly irrigated from the flow of the streams of that State, and because of this abundance of land, as compared with the available water in the regular flow, it is always in order to look for storage opportunities along a stream. It sometimes happens that a natural storage basin may be found near enough to the head of the farming area to make the storage dam serve the purposes of a diversion dam as well. Diversion is then a question of conduits and gates for conducting the water past the dam and into the canals. As these conduits and gates are a necessary part of the storage dam in any case, they need not be noticed in detail in this paper.

Occasionally, water has been diverted from sandy-bottomed streams of steep grade by collecting it in perforated pipes or timbered conduits buried in the stream bed and brought to the surface by extending at a gentle grade far enough down stream to gain the necessary elevation.

Heavy floods will pass over an intake of this kind without damage, and a constant small flow can sometimes be obtained from the ground where the stream is ordinarily dry at the surface.

Fig. 5 illustrates a plan recently devised by the writer for a case on the Gila River where the area of the land under the canal would not justify the expense of a permanent structure extending entirely across the river bottom, which at this place is wide. The canal company has heretofore maintained at its head-works a brush and earth dam which has been carried away by small rises of water several times in a season. The great loss from this condition, as mentioned previously, is to growing crops which are deprived of water. The main object of the present plan is to provide an escape for the small summer rises in the river (say from 2 000 to 10 000 sec-ft.), which are of short duration, without at the same time losing the head necessary to keep the canal running. As the flow during winter floods, and an occasional summer flood, is very much greater than the foregoing figures, the dam, which raises the water only 3½ ft., is designed to go out automatically when the danger level is reached, thus largely increasing the discharge area and leaving no obstruction in the channel above the concrete foundations, which rise only to the natural bottom level of the river. When a flood has thrown down the dam, it will be comparatively easy to set up the steel frames again and put in place the planks which make the weir. The concrete foundation is on a bed-rock footing. It is expected that the earth dike, which closes the gap from the automatic dam to the farther shore, will be broken through by occasional extreme floods. It should be planted with willows or cottonwoods and Bermuda grass to help bind it together.

For the purpose of springing the latch which holds up the first 10-ft. section of the dam, it is proposed to have a protected well, connected with the river water, in the masonry of the abutment at the shore end, in which is enclosed a heavy float operating the necessary lever. The fall of the first frame is designed to trip the latch of the next, and so on until the fall of the dam is complete.

The individual planks of the weir are attached to a wire cable, the end of which is free at the point where the sections are first released, the other end being anchored to the opposite end pier. This precaution is taken to save as many pieces of this timber as possible

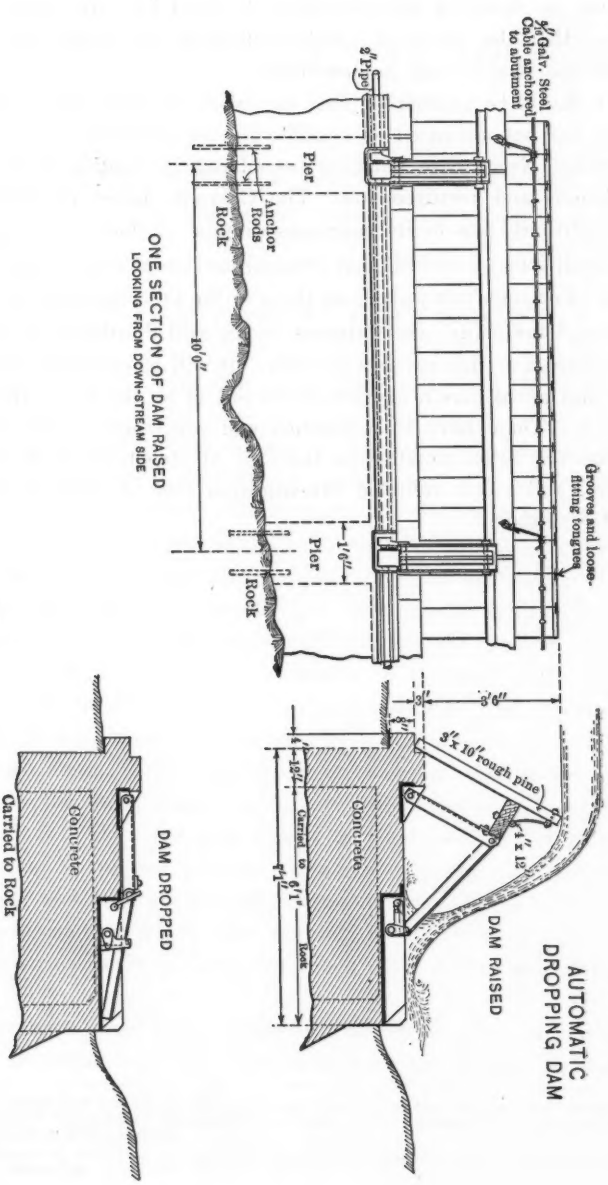
⅜" Galv. Steel Cable anchored to abutment
8" Pipe
Pier
Anchor Rods
Rock
10'0"
ONE SECTION OF DAM RAISED
LOOKING FROM DOWN-STREAM SIDE
Grooves and loose-fitting tongues
1'6"
Pier
Rock
3'6"
3" x 10" rough pine
1" x 12"
Concrete
12½"
Carried to
Lock
6'1"
DAM RAISED
AUTOMATIC DROPPING DAM
Concrete
Carried to Rock
DAM DROPPED

Fig. 5.

for use in replacing the dam after the flood has run down. It is expected that the string of planks will float and swing over to shore, where the lumber may be recovered.

It should be understood that the types of works mentioned in this paper are not advanced as suitable for all conditions to be found in Arizona. Every case must be considered in relation to its peculiar conditions and requirements. The Colorado River presents a series of conditions, due to its enormous volume of flood discharge, and to the conditions of its bed, that demand particular and unclassified solutions. For such water-sheds as those of the Gila and Salt Rivers, however, and for many other streams in the arid Southwest, it is believed the contents of this paper will apply. It will be observed that the best examples noted herein are structures put in by the U. S. Reclamation Service. These have been planned and constructed with an amount of care not often equalled in the case of structures built by private parties, and much valuable investigation and observation have been devoted to them.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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STEEL STRESSES IN FLAT SLABS.*

By Dr. H. T. Eddy.†

The question of the percentage of reinforcement in flat slabs is important economically as well as theoretically. It is, moreover, a question respecting which there seems to be some misconception as to the principles which should be applied. Shall the concrete and steel be balanced as perfectly as possible; or, if not, which shall be made the stronger? The uninformed have usually insisted on over-reinforcement, so that the concrete would be the weaker. If it is possible to determine the actual stresses in the steel with precision, there is no doubt that slabs should be designed so that the steel will give way first. Concrete is an unreliable and fragile material compared with steel. Under-reinforcement, however, will make security depend on the steel, which has known properties capable of precise computation. When the steel begins to yield, that action will cause the concrete to fracture to some extent, but the structure will exhibit toughness, and will not be subject to sudden collapse. In case of over-reinforcement, where the concrete is crushed and gives way first, the structure is in danger of much more sudden and unforeseen failure.

That these conclusions coincide with those of economic design appears from the following consideration of the stresses in steel and

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

† Professor of Mathematics and Mechanics, Emeritus, College of Engineering, University of Minnesota.

concrete, as given by Turneaure and Maurer's equations for the equal resisting moments of steel and concrete per unit of width of slab regarded as a beam, namely,

$$M_s = f_s\, Ajd = f_s\, pjd^2, \text{ and } M_c = \frac{1}{2}\, f_c\, kjd^2,$$

in which $A = pd$ is the cross-section of the reinforcement per unit of width of slab, kd = the depth of the neutral axis, and jd = the depth of the steel below the center of compression of the concrete. As these moments are equal, the ratio of the stresses in steel and concrete is,

$$\frac{f_s}{f_c} = \frac{k}{2\,p},$$

in which p is the steel ratio of reinforcement. Using Turneaure and Maurer's tabular values of k, in case $n = \frac{E_s}{E_c} = 15$, and assuming that $f_c = 500$ lb. per sq. in., and $f_s = 16\,000$ lb. per sq. in., the values of stresses in steel and concrete, respectively, shown in Table 1, are found.

TABLE 1.

$n = 15$.		$\frac{f_s}{f_c} = \frac{k}{2\,p}$.	For $f_c = 500$ $f_s = \frac{250\,k}{p}$.	For $f_s = 16\,000$ $f_c = \frac{32\,000\,p}{k}$.
p.	k.			
0.0025	0.25	50	25 000	320
0.005	0.32	32	16 000	500
0.010	0.42	21	10 500	762
0.015	0.48	16	8 000	1 000
0.020	0.525	13	6 550	1 220

Table 1 shows how small the percentage of reinforcement should be (less than one-half of 1%) in order that, in case of under-reinforcement, the steel should be used with reasonable economy; also, in case of over-reinforcement, how great the stresses in the concrete become in order to develop the strength of the steel. Every consideration of correct design, safety, and economy seems to demand a very low percentage of steel, verging on under-reinforcement, rather than on any attempt at over-reinforcement.

If these principles are adopted as controlling the design of flat slabs, the all-important and crucial question is that of unit stresses in the steel. The writer hopes that the following discussions of tests

for actual stresses in several large buildings, as compared with computed theoretical stresses, may assist in supplying a satisfactory basis for computation and design such as has not been available hitherto.

The Northwestern Glass Company Building.

The mushroom, flat slab floor in the Northwestern Glass Company Building, in Minneapolis, Minn., was tested by Mr. F. R. McMillan, on May 12th to 24th, 1913. The pressure for immediate occupancy of the building by the owners was such as to compel the completion of the test at an undesirably early date after the slab, which had been frozen all winter, had sweated out, and had had an opportunity to commence to become cured. It is estimated that the condition of the slab at the beginning of the test was perhaps such as would be expected at the end of a period of 45 days favorable for curing.

Appendix A contains the particulars of the test conducted by Mr. McMillan, whose results are used in this discussion. The equations applied in the computations may be found in the writer's book, "The Theory of Flat Slabs."*

The maximum load on Panel D amounted approximately to 185 000 lb. This was placed on the four quarters of the panel, with open passageways, 1.5 ft. wide, across the middle of the panel each way, and with open spaces around the columns at each corner arranged so that only 181.5 sq. ft. of the total panel area of 16 by 17 ft. = 272 sq. ft., were actually covered by the loading. With such an arrangement of the loading it was difficult to determine just what should be assumed as the equivalent uniform load. The difficulty is increased by the fact that, no matter what load is placed on the mushroom heads, it will have little effect in increasing either stresses or deflections. Consequently, attention must be given principally to the equivalence of the loading not located over the heads.

The open passageways decreased the effectiveness of the actual loading, and the open spaces over the heads increased its effectiveness in a very complicated manner when one also considers their stiffness relative to the remainder of the panel area. It will be evident, however, that, on the whole, the loading as actually applied, was more effective in producing deflections and stresses than if the total actual

* The numbers of the equations in this paper are the same as those in "The Theory of Flat Slabs."

a value of W, somewhat different from that needed in computing stresses, might be required.

The writer has proposed the following equation

$$f_s = \frac{W\,L}{175\,d_1\,A} \qquad (34)$$

as giving the limiting unit stress in the rods of the side belts. In the present case this gives, for the rods of the short side belts,

$$f_s = \frac{200\,000 \times 16 \times 12}{175 \times 7.31 \times 1.6567} = 18\,000 \text{ lb. per sq. in.}$$

Two of the observed readings, on Fig. 4, namely, those at 3 and 10, amounted to 17 000 lb. per sq. in., and it is probable that if all the panels of the slab had been loaded at once with a uniform load of $W = 200\,000$, all these side rods would have shown nearly or quite the foregoing computed value of f_s. That this is so seems to be clear from the fact that these readings were taken in belts adjacent to panels which were not loaded, and that the stress at 10 under Load 3, when the total load was placed nearly equally on four panels instead of on two, was not much smaller than under Load 5. The changes in the stresses at 7, 8, and 9, between Load 3 and Load 5, lead to the

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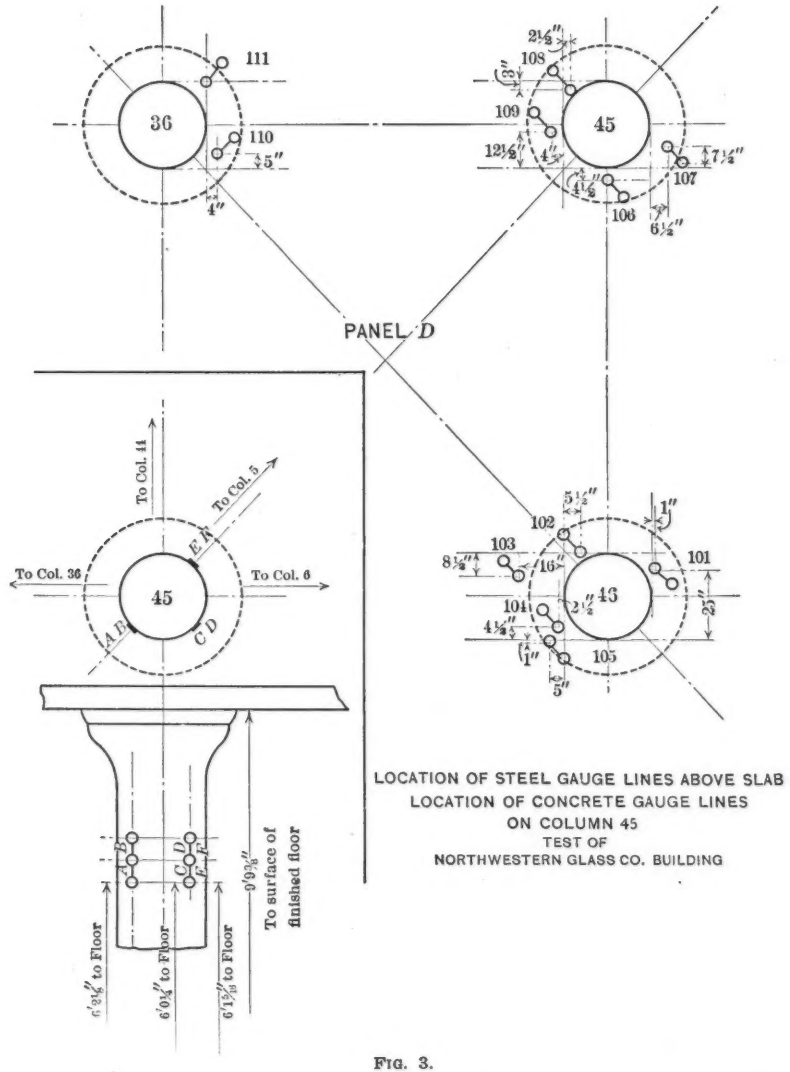
It will be seen, by comparing the observed stresses in the short side belt with those in the long side belt, that the latter were somewhat less than the former. This is the only kind of divergence between Equation (34) and experimental results which has come to the writer's attention. The divergence in this case appears to be due primarily to the nearness of these panels to the wall and to their position relative thereto.

Suppose a slab to be loaded in such a manner that the entire load was placed in continuous narrow strips, extending entirely across the middle of each panel, in a line parallel to the wall. These strips might be regarded as concentrated central loads on a wide beam having one end at the wall, with a depression or trough between the wall and the first row of columns parallel to it, another between the first and second rows, etc. These depressions would be much more pronounced than any crossing them perpendicularly to the wall.

The wall and Load 5 evidently had an effect of this kind, which caused the stresses in the short side belts perpendicular to the wall to exceed those in the long side belts.



LOCATION OF STEEL GAUGE LINES ABOVE SLAB
LOCATION OF CONCRETE GAUGE LINES
ON COLUMN 45
TEST OF
NORTHWESTERN GLASS CO. BUILDING

FIG. 3.

Indeed, slab action in general may be described partly as the attempted mechanical superposition of one set of parallel depressions and elevations on another set of similar corrugations at right angles to them. Such sets mutually support each other and give rise to slab

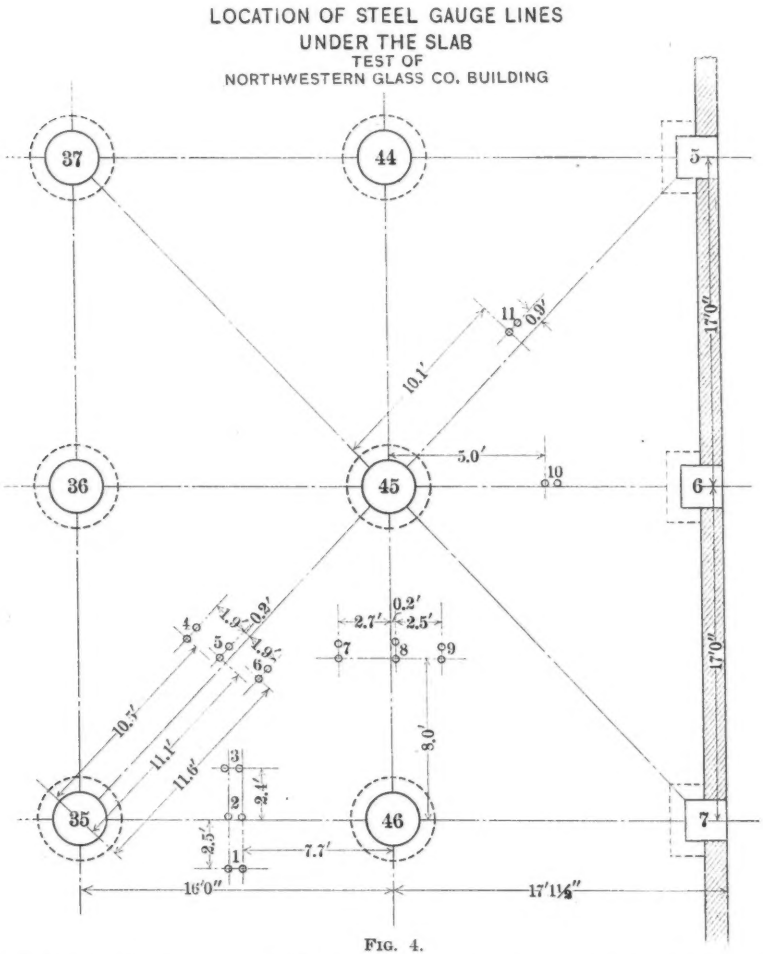


FIG. 4.

action, but any action which interferes with and disturbs the regularity of this superposition needs consideration. A wall support is such an interference, for it completely destroys the regular sequence of depressions and elevations at the end of one set and thus intensifies

those in the other set. The case just discussed is evidently of this character.

The writer has shown* that the stresses to be expected in the diagonals at the center of the panels are somewhat less than those in the middle of the side belts when all the panels of a slab are equally loaded.

By Equation (52) the unit stress in the middle rod of the diagonal at the center of the panel is

$$f_s = \frac{C_1 W L_1}{256 j d_2 A_1} = \frac{200\,000 \times 204}{256 \times 0.89 \times 6.94 \times 1.6567} = 15\,570 \text{ lb. per sq. in.}$$

The observed value at 5 in Panel D was 14 200 lb., which was larger than that in Rods 4 and 6 on each side of it, as required by theory. The stress at 11 in the wall panel, C, was larger, and amounted to 20 500 lb., which probably was due to the cantilever action at the wall being less than that exerted by the usual column heads.

Consider now the stress in the belt rods near the edge of the cap. In order to do this it is necessary to treat them somewhat more in detail than has been done in the writer's book, where the entire right section of diagonal and side belts has been regarded as unaffected by the mass of the cap, which forms a large boss, integral with the slab, and, as far as it extends, renders the belt much less extensible and compressible than it would be without it. The edges of each belt are so far from the edge of the cap as to permit of the assumption that the rods at the edge have the same stresses as they would were the cap not present; but those rods which are tangent to the cap will have their elongations and stresses reduced thereby, and the rods beside the cap and intermediate between these and the edges of the belts will have their stresses reduced in proportion to their proximity to the cap.

The case is different, however, with rods which cross the edge of the cap nearly perpendicularly. Although that part of the length of the rod which is inside the cap has its elongation prevented by the mass of the cap, the part outside must have its elongation correspondingly increased to compensate for this loss, and, on the whole, be equal to that of the rods beside it. The stresses in the middle rods of each belt, consequently, are increased abnormally for this reason just as it leaves the cap. Instead of attempting to determine this increase by

* "The Theory of Flat Slabs."

some intricate investigation, it will simply be assumed that the stress at this point in the middle rod does not exceed that in the outside rod of the belt at a point opposite the center of the cap. This must be very nearly the fact, and the stress in the outside rod at this point, as affected by the size of the cap, will now be considered.

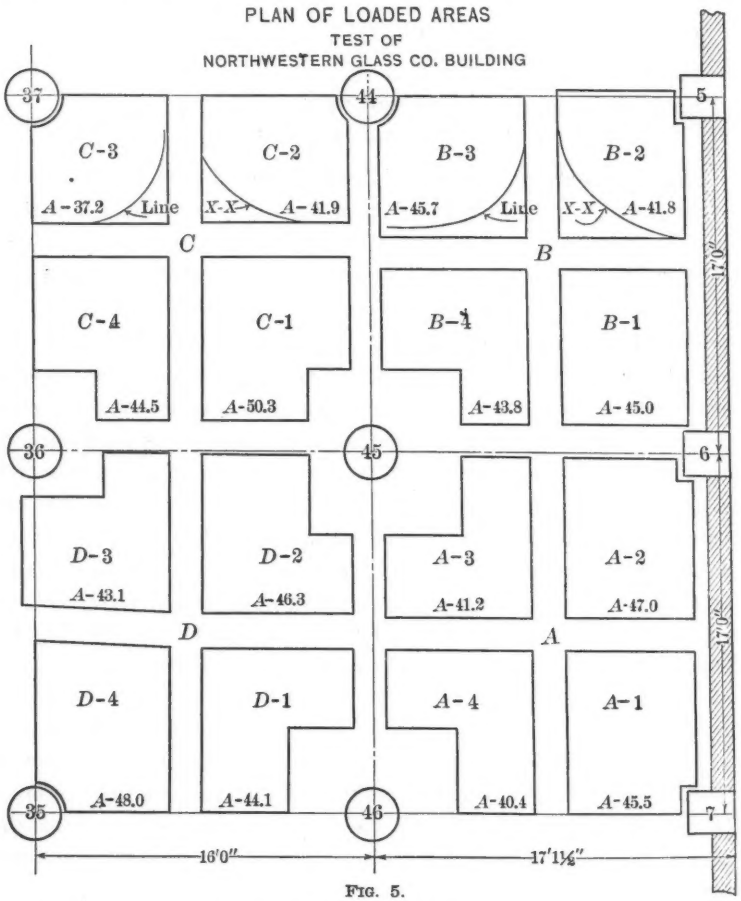
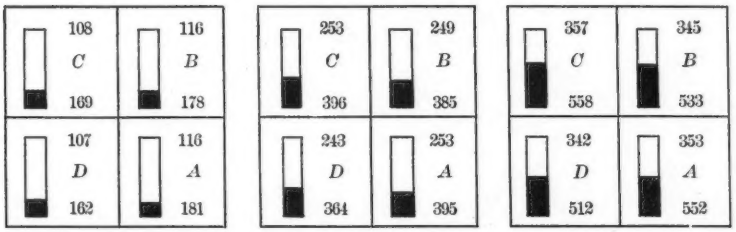


FIG. 5.

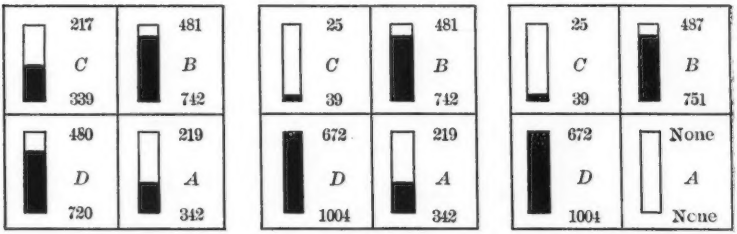
As the cap is integral with the slab and may be taken as horizontal at the edge of the cap instead of at the center of the column, as has been assumed in the writer's equations, the position of the points of inflection or contraflexure of the rods will be nearer the center of the

DETAIL OF LOADING
TEST OF
NORTHWESTERN GLASS CO. BUILDING



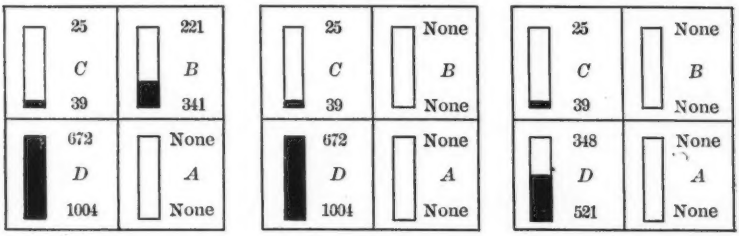
Load 1 2 3

Shaded rectangles show relative loading on different panels at all loads



4 4a 5

Load 9: All load removed except 7000 lb. in Panels *C* and *B*

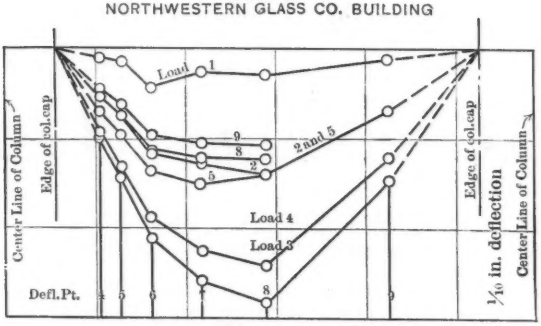


6 7 8

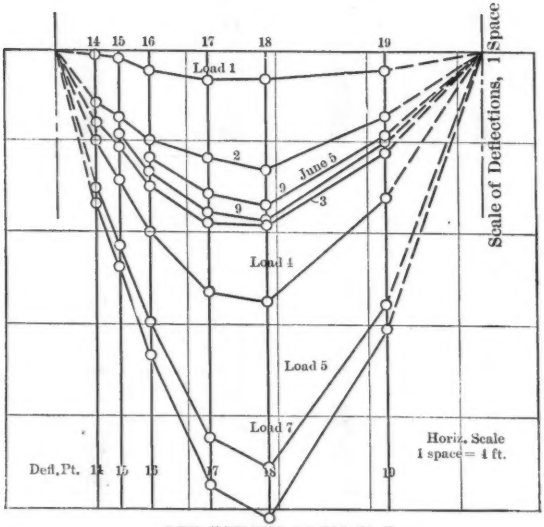
Upper figures in each panel show load per square foot over the whole panel area and lower figures over the actual loaded area.

FIG. 6.

DIAGRAMS OF DEFLECTIONS
ALONG DIAGONALS OF PANELS *A* AND *D*
TEST OF
NORTHWESTERN GLASS CO. BUILDING



DEFLECTIONS IN PANEL *A*



DEFLECTIONS IN PANEL *D*

FIG. 7.

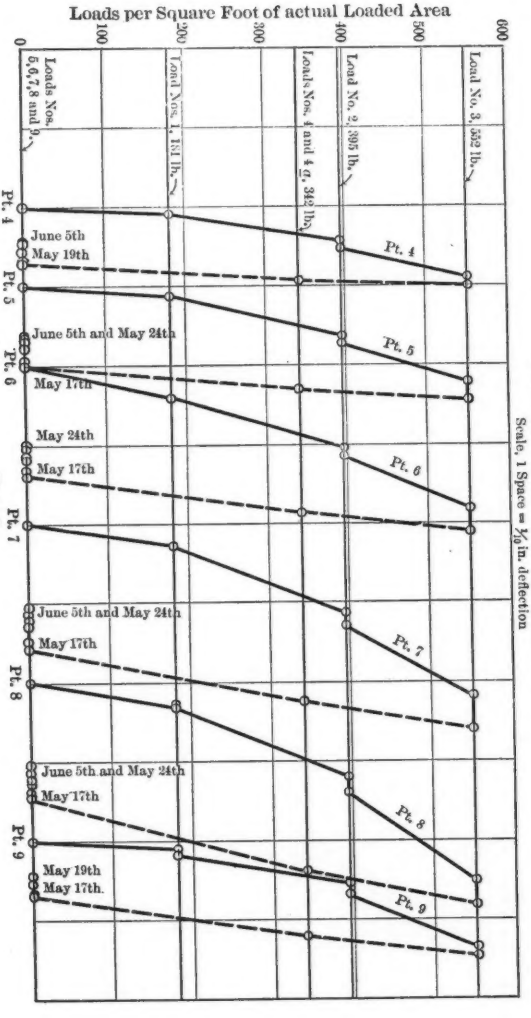

DIAGRAMS OF DEFLECTIONS UNDER PANEL A
POINTS 4 TO 9, INCLUSIVE
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 Space = 1/16 in. deflection
Loads per Square Foot of actual Loaded Area
0
100
200
300
400
500
600
Loads Nos. 5, 6, 7, 8 and 9.
Load Nos. 1, 151 lbs.
Loads Nos. 4 and 4a, 342 lbs.
Load No. 2, 395 lbs.
Load No. 3, 552 lbs.
Pt. 4
Pt. 5
Pt. 6
Pt. 7
Pt. 8
Pt. 9
June 5th
May 19th
June 5th and May 24th
May 17th
May 24th
May 17th
June 5th and May 24th
May 17th
June 5th and May 24th
May 17th
May 19th
May 17th.

Fig. 8.

DIAGRAM OF DEFLECTIONS UNDER PANEL *B*,
POINTS 1, 2, 3, 28, 29, AND 30,
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 Space = 1/10 in deflection

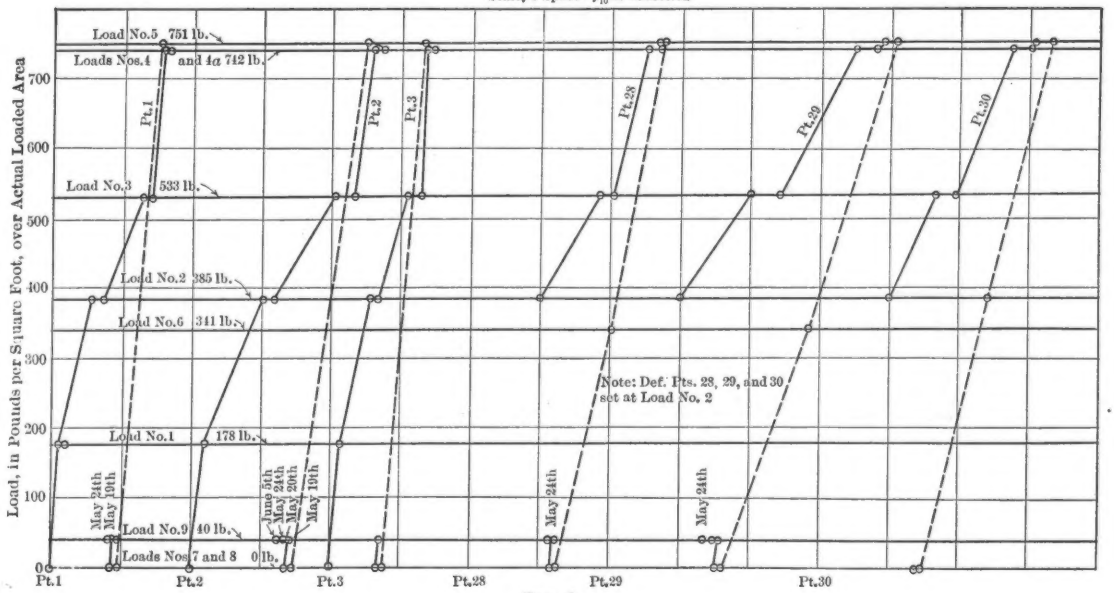


FIG. 9.

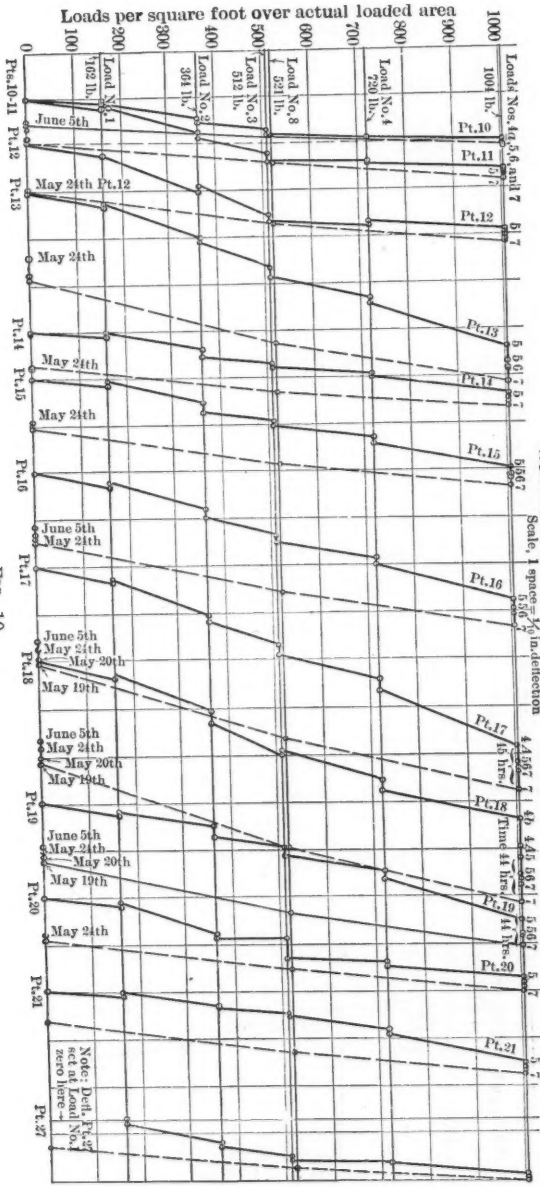


DIAGRAM OF DEFLECTIONS UNDER PANEL D
POINTS 10 TO 21, INCL., AND POINT 27
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 space = 1/16 in. deflection
Loads per square foot over actual loaded area
0
100
200
300
400
500
600
700
800
900
1000
Loads Nos. 4a, 5, 6, and 7
1004 lb.
Load No. 4
720 lb.
Load No. 8
521 lb.
Load No. 3
512 lb.
Load No. 2
364 lb.
Load No. 1
162 lb.
Pts. 10-11
Pt. 12
Pt. 13
Pt. 14
Pt. 15
Pt. 16
Pt. 17
Pt. 18
Pt. 19
Pt. 20
Pt. 21
Pt. 27
Pt. 10
Pt. 11
Pt. 12
Pt. 13
Pt. 14
Pt. 15
Pt. 16
Pt. 17
Pt. 18
Pt. 19
Pt. 20
Pt. 21
June 5th
May 24th
May 20th
May 19th
Time 44 hrs.
Note: Defl. Pt. 27 set at Load No. 1 zero here

FIG. 10.

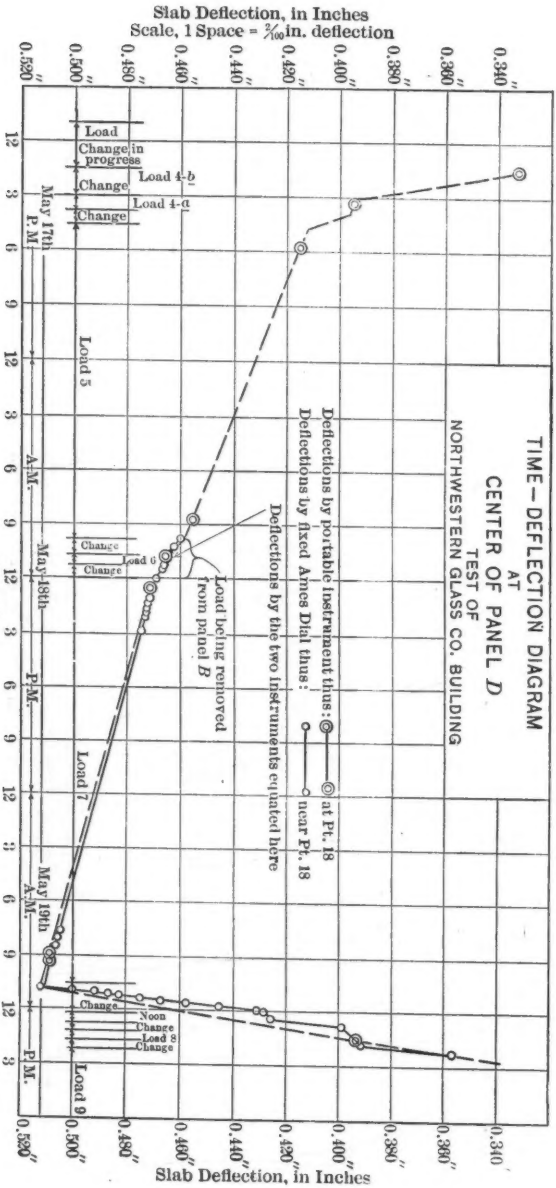

Slab Deflection, in Inches
Scale, 1 Space = 1/100 in. deflection
TIME—DEFLECTION DIAGRAM
AT
CENTER OF PANEL D
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Deflections by portable instrument thus: at Pt. 18
Deflections by fixed Ames Dial thus: near Pt. 18
Deflections by the two instruments equated here
Load being removed from panel B
Load 4-b
Load 4-a
Load 5
Load 6
Load 7
Load 8
Load 9
Change
Load in progress
Noon
May 17th P.M.
A.M.
May 18th
P.M.
May 19th
A.M.
P.M.
0.340"
0.360"
0.380"
0.400"
0.420"
0.440"
0.460"
0.480"
0.500"
0.520"
Slab Deflection, in Inches

FIG. 11.

span than has been assumed in those equations, when account is taken of the size of the cap. In Equation (42), a stress equation, the position of the points of inflection of the side belts is determined to be at distances $\pm \frac{a}{\sqrt{3}}$ and $\pm \frac{b}{\sqrt{3}}$ from the middle of the span, because the belts are horizontal at the middle of the span and over the column centers. Now if, instead, the belt is horizontal at the middle, and at the edge of the cap, which, for convenience, is assumed to be square, the points of inflection will be at a distance from the mid-span of one-half the clear span between the caps multiplied by $\frac{1}{3}\sqrt{3} = 0.577$. Subsequently, this will be found to be in agreement with experimental determinations.

Instead of Equation (42), we shall have, for the limiting stress in the side belt rods opposite the column center

$$f_s = \frac{W L_1 (L_1 + L_2)}{800 d_3 A_1 L_2}\left[\frac{3 L_1^2}{B_1^2} - 1\right] \ldots\ldots\ldots\ldots (a)$$

To find f_s at any other point, put 2_x in place of L_1. This equation gives larger unit stresses in the side belt rods of the head than Equation (42), but will itself need to be reduced somewhat in case the belt under consideration has an unusual number of laps in the head. However, it leaves the unit stresses determined by Equation (34) unchanged. The reason for these larger stresses is the longer cantilever span due to the location of points of inflection.

As the law of the distribution of stresses in the rods of a diagonal belt differs from that in a side belt, where all the rods have an unequal stress, the writer will obtain an analogous value of the stress in the rods of a diagonal belt. Starting with Equation (39), which refers to the part of the panel forming the column head, and transforming to axes, x' and y', along the diagonals, as was done in Equation (47), an expression will be obtained which is identical with Equation (47), except that g will be replaced by $a + b$.* Hence, in place of Equation (49), we have for this area:

$$\pm e' = id\,\frac{\delta^2 z}{\delta x'^2} = \frac{(1 - K^2)\, q\, (a + b)}{24\, E\, j\, d_3\, \Sigma\, A}\left[a^2 + b^2 - 3\,(x'^2 + y'^2)\right]$$

*There is a misprint in the last term of Equation (47) in the writer's book, and $x'^2 + y'^2$ should be substituted for $x'^2 y'^2$.

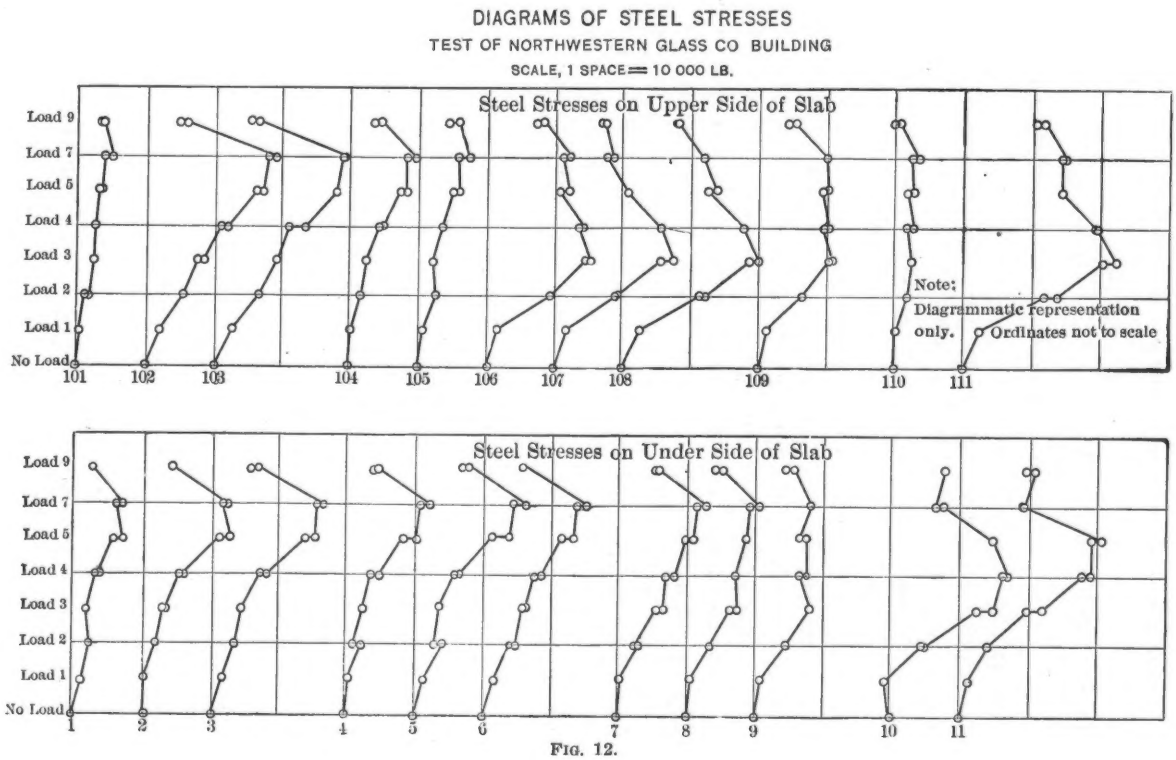

DIAGRAMS OF STEEL STRESSES
TEST OF NORTHWESTERN GLASS CO BUILDING
SCALE, 1 SPACE = 10 000 LB.
Steel Stresses on Upper Side of Slab
Load 9
Load 7
Load 5
Load 4
Load 3
Load 2
Load 1
No Load
101
102
103
104
105
106
107
108
109
110
111
Note:
Diagrammatic representation only. Ordinates not to scale
Steel Stresses on Under Side of Slab
Load 9
Load 7
Load 5
Load 4
Load 3
Load 2
Load 1
No Load
1
2
3
4
5
6
7
8
9
10
11

FIG. 12.

In this, by Equation (41), $j \Sigma A = 6 A_1$. Introduce the effect of the size of the cap in the same manner as in Equation (a), derived for side belts, and apply it to find the stress at the edge of the belt opposite the column center, where:

$$4 x'^2 = L^2 = L_1^2 + L_2^2 = 4 (a^2 + b^2), 4 y'^2 = g^2$$

$$4 (x'^2 + y'^2) = L^2 + g^2 = L'^2, \text{ and,}$$

$$C_1 = \frac{1}{4}\left(\frac{L_1}{L_2} + 1\right)\left(1 + \frac{L_2^2}{L_1^2}\right) = 1, \text{ nearly, then,}$$

$$f_s = Ee' = \frac{C_1 W L_1}{400 d_3 A_1}\left(\frac{3 L_2^2}{B'^2} - 1\right) \ldots\ldots\ldots\ldots\ldots\ldots (b)$$

where B' is the clear span along the diagonal between the caps. This equation gives the stress in the rod at the edge of the diagonal belt opposite the column center, and this is assumed also to give the value of the unit stress in the middle rod of the belt just outside the circular cap. To find the stress at any other point, x' y', of the diagonal belt in the head, substitute $4 (x'^2 + y'^2)$ for L'^2.

It will be noticed that in the central area of the panel, where moments are positive, the stress in the diagonal rods is greatest in the middle rod, but, in the column heads, where moments are negative, the stress is greatest in the rods at the edge of the belt.

The value of the greatest tensile stress at the edge of a direct or diagonal belt, found from Equations (a) or (b), however, is computed on the assumption that the steel in the slab is practically at the same depth as the rods nearer the center of the same belt. Such, however, is not ordinarily the case. The edges of the belts near the cap lie at a somewhat lower level than the remainder of the belt, and, being nearer the neutral axis, the rods at the edges, for that reason, will not be subject to as large a stress as is computed from Equations (a) or (b), and may not have as large stresses as rods somewhat nearer the center.

In the rods of the diagonal belts, points of inflection and points of equal stress lie on circles with centers at the center of the panel, so that the points of inflection of the side belts which lie on lines perpendicular to the sides do not coincide with the points of inflection of the diagonal rods.

In this test no observations were made on the rods of the side belts by which any comparison can be made with Equation (a), but several points on the diagonals near the edges of the caps were ob-

served, which may be estimated by Equation (*b*), though no observations were made on the rods near the edges of the diagonal belts.

Under Load 3 it will be assumed that each of the four panels was loaded with what would be equivalent to a uniformly distributed total

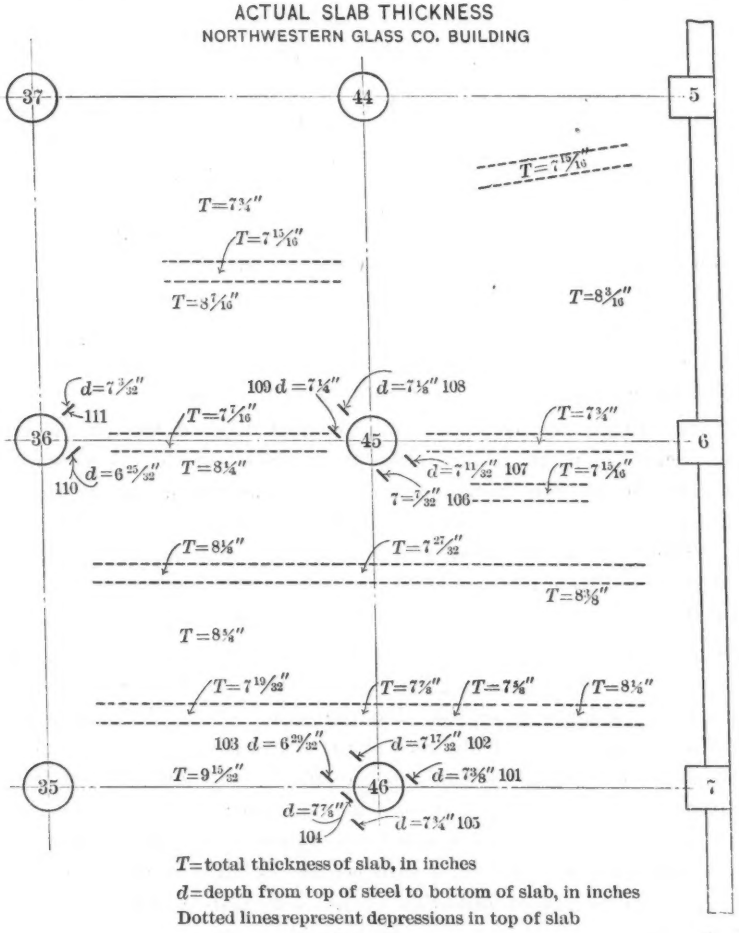


FIG. 13.

load of $W = 106\,250$ lb. By Equation (*b*) the limiting value of the unit stress in the outer rod is

$$f_s = \frac{106\,250 \times 204}{400 \times 6.5 \times 1.6567}\left(\frac{3 \times 86\,049}{60\,025} - 1\right) = 17\,000 \text{ lb. per sq. in.}$$

This is to be compared with the stresses observed on Fig. 3, most especially at 107 and 108, around the central cap of the loaded area, which were, respectively, 17 500 and 20 000 lb. Incidentally, comparisons should be made with observations at 102 and 111, at the caps on the edge of the loaded area.

Good agreement of the computed with the observed results is hardly to be expected in this case, because, just as the concrete was about to be poured, it was discovered that the radial elbow rods, by some mistake, had not been bent at quite the correct angle to correspond with the thickness of the slab, and it was necessary to pry the ends of the rods upward forcibly and hold them in this position by blocks under their extremities, thus putting them under considerable initial

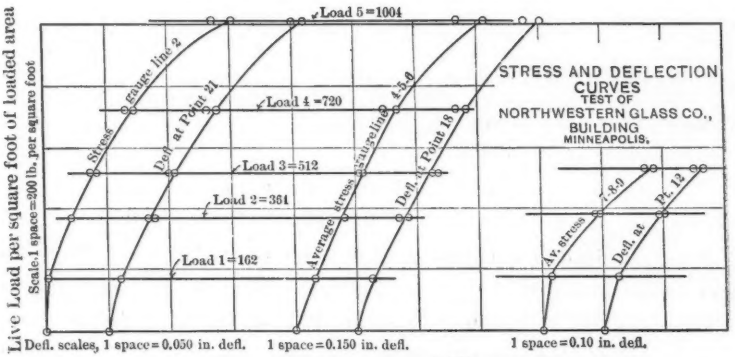


FIG. 14.

bending stress. The relief of this bending by the test load would influence the observations to some extent, and tend to make the observed values of f_s in the slab rods larger than those computed. At 111, Column 36, on the edge of the loaded area, the abnormal value of 22 400 lb. was reached under Load 3. Load 5 shows no such large increase over Load 3 of observed stress in the slab rods of the column heads in Panel D as was to be expected by practically doubling the loading. This fact is apparently inexplicable.

By reason of the disarrangement of the stresses due to initial bending, as mentioned, it seems to be impossible to trace the stresses in the slab rods in greater detail in this test than has been done already, where a satisfactory concordant maximum stress in the slab rods has

been reached. Consequently, a comparison of the computed with the observed deflections will now be considered. By Equation (71)

$$D_2 = \frac{W L_2 L_1^2}{4.4 \times 10^{10} A d_2^2}$$

in which $d_2 = 8 - 0.5 - 0.5625 = 6.9375$ in.

Hence $$D_2 = \frac{200\,000 \times 16 \times (17)^2 \times 1\,728}{4.4 \times 10^{10} \times 1.6567\ (6.9375)^2} = 0.4555 \text{ in.}$$

This agrees precisely with the value observed at 9 A. M. on May 18th under Point 18 at the center of Panel D. Practically the same deflection was found at the adjacent Point 17.

By Equations (61), (54), and (58), the deflection at the middle of the side belt is

$$D_1 = \frac{W L_1^3}{10^{10} A}\left[\frac{1}{10.7 d_1^2} + \frac{\frac{L_1}{L_2} + 1}{60 d_3^2}\right]$$

in which $d_1 = 7.31$ in. and $d_3 = 6.5$ in.

The values of the deflections are:

At Point 12: computed, 0.27 in., observed, 0.22 in.
" " 20 " 0.25 in., " 0.20 in.

The magnitude of the observed deflections in the side belts is less than those computed, for the same reason that the stresses are less, as before stated, although it is clear that the observed values would possibly reach those computed in case all the panels were loaded equally. This last remark is confirmed by considering the deflection under Load 3, which was nearly the same on all four panels, and equivalent to 106 250 lb., uniformly distributed, as found previously. The computed deflections would then be somewhat more than one-half of those already computed, and may be compared with the observed values, as follows:

At Point 12: computed, 0.15 in., observed, 0.16 in.
" " 20 " 0.13 in., " 0.13 in.

From which it may be concluded that the computed results would not be less than those observed if all the panels were equally loaded.

It seems clear that the continued increase of the deflection at Point 18 at the center of Panel D under Load 7 was influenced somewhat by the removal of the loading in Panel B, and that changes in the de-

FIG. 15.—SLAB REINFORCEMENT, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

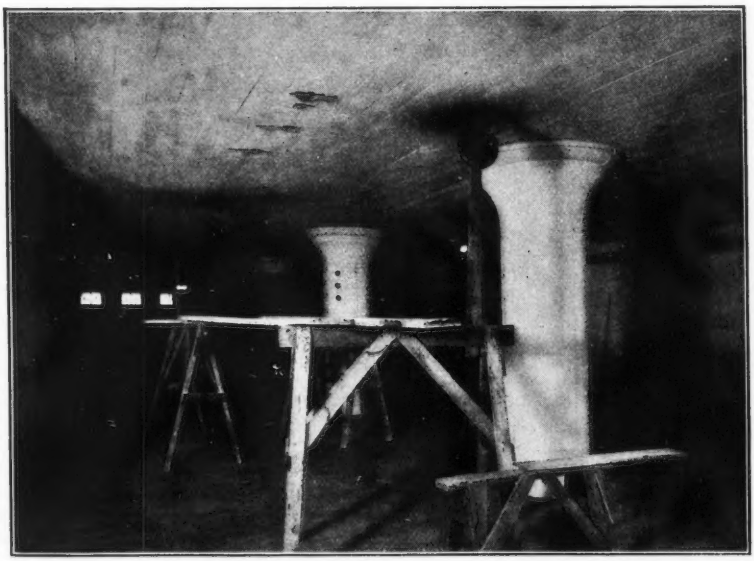


FIG. 16.—UNDER SIDE OF TEST PANELS, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

FIG. 17.—UNIFORM LOAD OF 400 LB. PER SQ. FT. OVER FOUR PANELS. NORTHWESTERN GLASS COMPANY BUILDING.

FIG. 18.—TEST OF NORTHWESTERN GLASS COMPANY'S BUILDING BY MINNEAPOLIS BUILDING DEPARTMENT.

flections of adjacent panels may be looked for along the prolongation of diagonals rather than across the sides of the panels. This is perhaps the first test of a mushroom slab in which it has been possible to discover interaction of panels. Possibly, however, this may be due to the initial stresses in the radial steel previously mentioned.

Fig. 7 is of interest as showing the dissymmetry in the right and left half spans of the diagonals in Panels A and D. In A the supporting action of the wall along the entire length of the side evidently prevented deflection to some extent in the half span nearest to it, so that Point 9 was deflected less than the corresponding point in the other half span. In Panel D Point 18 was deflected less than the corresponding Point 16, because all the slab rods are lapped over Column 35, and none over 45, as may be seen on Fig. 1.

Owing to the irregularities of loading and variations of stiffness due to the inclusion of the wall panels, as well as accidental variations of stiffness of construction due to initial bending stresses, and also the inequalities of laps, it is too much to expect formulas derived for entire uniformity of loading and equality of panels to apply more closely than these comparisons show.

The Deere and Webber Company Building.

The test of the Deere and Webber Building, in Minneapolis, Minn., was made on October 30th to November 5th, inclusive, 1910, by Mr. A. R. Lord.* The panel dimensions are 19 ft. 1 in. by 18 ft. 8 in., the slab is $9\frac{3}{16}$ in. thick, and the concrete was only 40 days old on October 30th. All the slab rods are $\frac{7}{16}$ in. round, with twelve rods in each side belt and fourteen in each diagonal. The head steel consists of the rectangular diamond frame with four rods extending entirely across it, which was at that time the typical form used by the Leonard Construction Company in such buildings. The column caps are 54 in. in diameter and the slab rods of all belts are 7.25 in. from center to center, so that the side belts may be taken to have a width of 80 in. and the diagonals nearly 95 in. The mean size of the cap is 0.24 L.

Various loads, uniformly distributed, were applied simultaneously to eight panels, which, with the omission of one corner panel, filled

*Reported by Mr. Lord in a paper printed in *Proceedings*, Nat. Assoc. of Cement Users, Philadelphia, Vol. VII, 1911.

out a square of nine panels. This gave opportunity to observe stresses in the slab rods in one panel at the center which was surrounded on all sides by equally loaded panels.

This discussion will refer mostly to the results observed when the greatest load was on all the eight panels at once. This load had a mean intensity of 350 lb. per sq. ft. The area of a panel is $356\frac{2}{3}$ sq. ft., and the total load $W = 350 \times 356\frac{2}{3} = 124\,678$ lb. The depth of the embedment of the various rods observed is given in the report, but, as the rods of a belt act together, it is not certain that the embedment of individual rods should be used in the formulas. It has been assumed that the fair mean values are

$$d_1 = 8.5 \text{ in., } d_2 = 8 \text{ in., and } d_3 = 7.6 \text{ in.}$$

Applying Equation (34) to compute the unit stress in the slab rods on the short side of the panel at Point 14, of Fig. 3, in Mr. Lord's paper, which is the point where the greatest stress was found in any slab rod of a side belt between two loaded panels,

$$f_s = \frac{124\,678 \times 224}{175 \times 8.5 \times 12 \times 0.15} = 10\,000 \text{ lb. per sq. in.}$$

The observed stress at this point was 10 400 lb. The unit stress observed at Points 39, 40, and 14*a*,* on the short side belts, were somewhat less, but this is a satisfactory determination of the greatest stress at the middle of a short side belt.

Applying Equation (34) to compute the unit stress in the long side belt at Points 108 and 109,

$$f_s = \frac{124\,678 \times 229}{175 \times 8.5 \times 12 \times 0.15} = 10\,220 \text{ lb. per sq. in.}$$

The mean observed result was 6 600 lb. It is probable that this result, observed between two loaded panels bounded by panels not loaded, would be less than on the same belt in the two loaded panels beside it, on the principle that if only three successive spans of a continuous beam of many spans be loaded, the middle one will show less stresses and deflections than the end spans.

This, together with the fact that the inflection lines were much nearer to the center of the panel, was probably the reason that the observed result was so small. Apparently, however, there is no reason

*These refer to Fig. 3 of Mr. Lord's paper.

for the greatest unit stress being less on the long side of the panel than on the short side, when the belts are equal and the panels are so nearly square. In this particular the results of the test seem to be inexplicable unless the two bulkheads which extend nearly across the slab parallel to the long sides of the panels, one of which crosses the short side at 14, have this effect.

A similar discrepancy was found between the observed stresses on the long and short sides of the panel in other tests. It is certain that provision should be made on this long side for stresses at least as great as those on the short side.

Applying Equation (52) to the computation of the unit stress in the rods of the diagonal belt at the center of the panel at Points 3, 7, 110, and 12*a*, gives

$$f_s = \frac{124\,678 \times 229}{256 \times 0.89 \times 14 \times 0.15 \times 8} = 7\,440 \text{ lb.}$$

There seems to be some discrepancy between the table of numerical values and the plotted maximum stress at Point 12*a*. In any case, 12*a* shows abnormally large readings compared with the others, but the computed value just found is greater than three of the observed values and greater than the mean of all four.

The diameter of the cap being $0.24L$, the clear span between the edges of the caps on the side belt is $0.76L$, and the least computed distance of the points of inflection along the middle of the side belt is this divided by $\sqrt{3}$, or $0.577 \times 0.76L = 0.44L$; and, consequently, the distance between the points of inflection across the column center is $0.56L$, which, as stated by Mr. Lord in a second paper,* is precisely the observed position of the points of inflection in the side belt of this slab.

Applying Equation (*a*) to the computation of the unit stress at Point 205, Fig. 3, of Mr. Lord's paper, on the edge of the long side belt, gives

$$f_s = \frac{124\,678 \times 229 \times 453}{800 \times 7.6 \times 13 \times 0.15 \times 224}\left[\frac{3 \times 52\,441}{30\,625} - 1\right] = 20\,150 \text{ lb.}$$

per sq. in.

The value observed at 205 was 20 000 lb., and slightly less at the symmetrical position, 208, also situated at the edge of a short side

* Also published in *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, 1911.

belt. The corresponding unit stress in a rod tangential to the edge of the cap at 206 was less, for the reason previously stated, being only 14 400 lb. at 206, and 12 300 lb. at 209.

Applying Equation (*b*) to the computation of the limiting unit stress in the rods at the edge of the diagonal belt opposite the column center gives

$$f_s = \frac{124\,678 \times 229}{400 \times 7.6 \times 13 \times 0.15}\left[\frac{3 \times 111\,642}{70\,756} - 1\right] = 22\,440 \text{ lb. per sq. in.}$$

All the observed stresses were somewhat less than 16 800 lb., both at 202, the outside rod, and at 204, the rod tangent to the cap, and 18 300, at 203, the rod lying between them. The outside rod, for the reason already stated, did not show quite the computed stress, but the observed stress in the center rod of the belt, where it crossed the edge of the cap at 207, was 23 400 lb., when all the panels were loaded, and 24 200 lb. when half the load had been removed from the outer panels.

Applying Equation (71) to the computation of the deflections at the center of the panel gives

$$D_2 = \frac{124\,678 \times 224 \times 229 \times 229}{4.4 \times 10^{10} \times 64 \times 14 \times 0.15} = 0.248 \text{ in.}$$

Point 37, Fig. 3,* was the only center point observed in a loaded panel wholly surrounded by loaded panels. The mean value of seven readings was 0.224. When half the load was removed from the outside panels the deflection increased to 0.246 in. The mean readings at the centers of adjacent panels were: Point 4, 0.291 in.; Point 8, 0.306 in.; Point 12, 0.271 in.; all these being somewhat larger because adjacent panels were not loaded.

Applying Equations (54), (58), and (61) to the computation of the deflections at the middle points of the side belts gives $D_1 = 0.1555$ for the long side, and $D_1 = 0.1456$ for the short side. The only reading at the middle point of a long side belt not continuous with or contiguous to an outside panel was at Point 17 where the mean of seven readings was 0.1387. The mean reading at Point 5, at the middle of the continuation of the same rod on the other side of Column 41, between two loaded panels, was 0.229. These points were situated similarly to the middle points of three successive equally loaded spans

* Of Mr. Lord's paper.

of a continuous beam, Point 17 being at the middle point of the middle span, and Point 5 at the middle of one of the end spans; the first deflection was somewhat less, and the other somewhat greater than the deflection when many spans were equally loaded. Point 41 was situated so that its mean deflection should fall between that at 17 and that at 5, which is the fact.

Point 38 was on a short side, on a location in the middle span of three spans like that of 17 on the long side, and the mean reading was 0.127; Point 15 was on a short side, as 5 was on a long side, in an end span of three spans, and the mean reading was 0.163. The computed deflection falls between these, as it should. The deflection at 42 should fall between the readings for Points 38 and 15, which is the fact. Point 11 was in practically the same relative position as 15, and had equal deflections. Other deflections at the middle of outside belts had about half the preceding deflections, influenced more or less by their continuities.

The interaction of successive spans across the column heads, by reason of continuity of spans, is evidently of considerable amount in this floor. This is doubtless due to lack of stiffness at the junction between columns and slab, which in this design is almost entirely without stiffening rods, such as the elbow rods in the mushroom system. The very striking agreement of these computations with the observations in this test show that in all essential particulars the action of the reinforcement is in principle identical with that for which the equations here applied were originally developed.

The Larkin Building.

A very extensive and painstaking test of the Larkin Building, in Chicago, was made by Mr. A. R. Lord.* The design of the slab reinforcement in this building was of the general four-way type used in the mushroom system, but the column heads omitted much of the steel ordinarily used for stiffening the head, and, in place of it, resistance to bending and shear was supplied by a depressed head or "drop" of concrete, 8 ft. square and 6.75 in. thick, placed at the top of each column, between the cap and the bottom of the slab. This was made integral with the slab and formed part of it, just above the cap. The panels are 20 ft. by 24 ft. 2 in., or $L_1 = 290$ in., and

*The results were presented by Mr. Lord in a paper at the Ninth General Convention of Cement Users. Extracts from this appear in the *Cement Era*, January, 1913, p. 52.

$L_2 = 240$ in. The thickness of the slab proper is 9 in., except at the drop, where it is 15.75 in. The diameter of the cap is 60 in. The width of the side belts is from 90 to 93 in., and of the diagonal belts from 105 to 108 in. All the slab rods are $\frac{1}{2}$-in. round, 13 in each short side belt, 22 in each long side belt, and 21 in each diagonal belt.

The floor was designed for a dead load of about 120 lb. per sq. ft., and a live load of from 225 to 250 lb., with a maximum test load of twice the sum of these, or actually 738 lb. per sq. ft. As the dead weight of the slab was acting at all times, the stresses tabulated in the report are those due to the observed elongations plus corrections due to the weight of the slab itself, which is included in the 738 lb. The deflections given, however, are those actually observed under the loads applied, or a maximum test load for deflections of about twice 250 lb., plus 120 lb., or a total actual maximum load of 618 lb. per sq. ft. Hence the total maximum load producing stress was $W = 738 \times 20 \times 24\frac{1}{3} = 356\,700$ lb. The load was applied, and the readings of stresses and deflections were made at several stages. At Stage 2, 130 lb. per sq. ft. was applied over a test area of five panels, three of them wall panels, and two adjacent to them, not wall panels. In Stage 3 the load on this area was increased to 250 lb. In Stage 4, only three panels were loaded, two of them wall panels and one adjacent interior panel, with 415 lb. per sq. ft. Finally, in Stage 5, the entire load of 618 lb. per sq. ft. was placed on two panels only, one wall panel, and an adjacent interior panel, and these were situated so that their short sides were contiguous.

Apply Equation (34) to the computation of the unit stress in the rods of this short side belt, in which, ordinarily, it would be found that $d = 8.25$ in., but among the test data this is stated to have a value of 8 in. in this case. Hence

$$f_s = \frac{356\,700 \times 240}{175 \times 8 \times 13 \times 0.196} = 24\,000 \text{ lb. per sq. in.}$$

The observed stress was 24 200 lb., which would be decreased somewhat, as noted previously, if adjacent panels were also equally loaded, as contemplated in Equation (34). Applying Equation (34) to the long side belt,

$$f_s = \frac{356\,700 \times 290}{175 \times 8 \times 22 \times 0.196} = 17\,000 \text{ lb. per sq. in.}$$

The observed stresses in the rods of a belt lying between a loaded panel and one not loaded is much less than between two loaded panels. The observed value in this location is given at 10 200 lb., but it is not stated whether this is a mean value or the greatest value for any rod of the belt, or the stress in the middle rod. From the figured location of the test lines, it is probably the latter. The computed results should be regarded as in most satisfactory agreement with the observations.

Applying Equation (52) to the computation of the unit stress in the middle rod of the diagonal belt at the center of the panel:

$$f_s = \frac{356\,700 \times 290}{256 \times 0.9 \times 7.75 \times 21 \times 0.196} = 14\,070 \text{ lb. per sq. in.}$$

in which 7.75 in. is taken as the mean distance of the center of the steel from the upper surface at the center of the panel. The given observed value was 12 900 lb., which differs from the computed by less than 10 per cent. It is not stated whether the observation was in the wall panel or in the interior panel, or whether it was an average or in the middle rod of the belt, but it was probably the latter. If so, the stress in the middle rod should be greatest. This result may be regarded as satisfactory, and very near what would be observed in the middle rod if the adjacent panels were equally loaded.

Before computing the stresses in the head steel at the edge of the cap and edge of the drop, it should be noted that all the stresses given in the general summary of stresses for the maximum load of 738 lb. are from 2½ to 3 times as great as for the design load, which is half as great. This, among other things, signifies that the value of the modulus of elasticity for the concrete was much less under the test load than was its initial value. Under the test load of 738 lb., there is no one of the columns entirely surrounded by loaded panels. The consequence is that the observed stresses in all the belts of the head will be much less than those computed, and if an attempt were made to compute the stresses due to the design load which was spread over all the panels around the column, where most of the readings on the head belts were evidently made, the observed stresses would have to be multiplied by a coefficient of 1.5 or more to make them comparable with the results at the test load, 738 lb.

Now write Equation (*a*) in the general form

$$f_s = \frac{W L_1 (L_1 + L_2)}{800 d_3 A_1 L_2} \left[\frac{3 (2x)^2}{B_1^2} - 1 \right] \ldots\ldots\ldots\ldots (a)$$

in order to compute the unit stress in the rods of the long side belt at the edge of the cap, where $2x = B_1 = 290 - 60 = 230$ in., and $d_3 = 14$ in. The drop is so thick as to obviate mostly the increase of stress in the center rod where it crosses the edge of the cap, and therefore no account is taken of such increase. This unit stress is found to be $f_s = 9\,460$ lb. per sq. in. The observed stress along this side at the edge of the load, of course, was considerably smaller, and is given as 7 000 lb.

Making a similar computation at the edge of the drop on this belt, where $B_1 = 230$ in., $2x = 290 - 96 = 194$ in., and $d_3 = 7$ in., it is found that $f_s = 14\,500$ lb. per sq. in. Along this side belt at the edge of the load the observed result was only 10 400. The diagram of gauge lines seems to show that these are readings observed along the middle rod of the belt. In that case the outside rod at the edge of the belt under a load over many panels would have shown greater unit stresses than those computed, or, in the writer's opinion, at least as large.

Applying Equation (*a*) to the computation of the unit stresses in the short side belt at the edge of the cap, by exchanging the subscripts, 1 and 2, and placing $2x = B_2 = 240 - 60 = 180$ in., $d_3 = 14$ in., and $A_2 = 13 \times 0.196$, gives $f_s = 10\,950$ lb. per sq. in. This belt crosses the cap from the unloaded into the loaded area which covers the two panels on each side of it. The tipping of the head relieves to some extent the stresses which would exist were all panels loaded, and the reading given is 7 300 lb.

Again, compute the unit stress at the edge of the drop in this short side belt by inserting in the equation just used $B_2 = 180$ in., $2x = 240 - 96 = 144$ in., $d_3 = 7$ in., and $f_s = 15\,300$ lb. per sq. in. At this point there is something abnormal and inexplicable in the observed results which, apparently, should be larger than those observed at the edge of the cap where the concrete is so thick. This contradictory reading is only 5 800 lb., and may be due to some bend in the rod, or other peculiarity, because the same thing appears at the smaller loads.

Writing Equation (*b*) in the general form

$$f_s = \frac{C_1\, W\, L_1}{400\, d_3\, A}\left[\frac{3\,(2\,x')^2}{B'^2} - 1\right] \ldots\ldots\ldots\ldots\ldots (b)$$

and computing the unit stress in the middle rod at the edge of the cap on the diagonal of 21 rods by increasing B' between the corners of the assumed square caps and placing

$$L = \sqrt{290^2 - 240^2} = 320 \text{ in.}, \quad B' = 320 - 85 = 235 \text{ in.},$$
$$2x' = 320 - 60 = 260 \text{ in.}, \quad f_s \text{ is found to be as follows:}$$

$f_s = 12\,480$ lb. per sq. in. The necessarily smaller observed value was 10 800 lb.

Apply the same equation to compute the unit stress in the middle rod of the diagonal belt at the edge of the drop, assumed to be 96 in. in diameter because the projecting corner should be disregarded, and we have $2x' = 320 - 96$ in., $B' = 235$ in., $d_3 = 7$ in. Hence $f_s = 16\,160$ lb. per sq. in. The smaller observed value was 14 200 lb.

Applying Equation (70) to compute the deflection at the center of the panel gives

$$D_2 = \frac{W\, L_2\, L_1^2}{10^{10}\, A_1}\left[\frac{1}{6.56\, d_2^2} + \frac{1}{12.5\, d_3^2}\right]$$

in which the total load producing the observed deflection was 618 lb. per sq. ft., or a total load of $W = 296\,850$ lb.

$$D_2 = \frac{296\,850 \times 240 \times 290^2}{10^{10} \times 22 \times 0.196}\left[\frac{1}{6.56 \times 7.75^2} + \frac{1}{12.5 \times 14^2}\right]$$

or $D_2 = 0.41$ in. The observed deflections at Points 10 and 16 were 0.40 in., an extraordinary agreement, under the circumstances, of reduction of steel in the head and displacement of inflection lines which produce effects that tend to balance each other.

Deflections at the middle of the long side may be computed by Equations (91), (54), and (58), as follows:

$$D_1 = \frac{296\,850 \times 290^3}{10^{10} \times 22 \times 0.196} \cdot \frac{1}{10.7 \times 8^2} \cdot \frac{2.2}{60 \times 14^2} = 0.276 \text{ in.}$$

The observed deflections were 0.21 in. at Point 15 and 0.26 in. at Point 17, on the edges of the loaded area, which values would be somewhat increased if the adjacent panels were also equally loaded. A similar computation at the middle of a short side belt gives results still more in excess of the value observed at Point 13, Fig. 3, of Mr. Lord's paper,

by reason of the shortness of the concave section between the points of inflection. The surprising thing is, not that such discrepancies exist between computed and observed results, but that equations derived for a flat slab differing so widely and in so many particulars from this one should nevertheless be applicable at all.

The writer's equations were derived for the case of side belts with total cross-sections proportional to their spans and with diagonals with intermediate cross-sections which, according to his theory, is the correct proportion. The Larkin slab, however, has the cross-sections of its side belts proportional to the cubes of their spans, which is a highly uneconomical arrangement of steel, as shown by the unit stresses observed in the test. The fact that these equations apply so closely shows that the use of the drop in stiffening the head is practically identical in action with the steel frame of the column head which it replaces.

In view of the unqualified assertion made in the extract from Mr. Lord's paper in the *Cement Era,* respecting the Larkin test, that "the deflection readings, compared with those of other tests, show that the resulting structure is stiffer and stronger than the straight flat slab as ordinarily designed, with anywhere the same amount of materials", it becomes of interest to submit such a straight flat slab to analysis for purposes of comparison, as the writer's formulas are applicable to such a slab with considerable accuracy.

It is to be noticed that deflection readings do not of themselves permit any conclusions whatever to be drawn as to the strength of a structure, but have reference to stiffness only, so that that half of the assertion referring to strength is, on the face of it, unwarranted and entirely unsupported by these readings, and, in fact, will be found to be incorrect, as the writer will proceed to show. As to deflections, it may be that a perfectly flat slab might have a deflection of perhaps not more than 10% in excess of this slab, with its bulky and unsightly drop at the head of each column. If that should possibly be the case, it is a trivial matter, and of no consequence compared with the very serious loss of strength involved in this construction.

The volume of the drop, which is 8 ft. square and 6.75 in. thick, is 62 200 cu. in. In case this quantity of concrete were used to increase the mean thickness of the slab, it would make a flat slab approximately 0.9 in. thicker than at present. Such a flat slab 9.9 in. thick through-

out, with belts of 0.5-in. round slab rods, having 19 rods on each short side, 19 in each diagonal belt, and 22 on each long side, would have slightly less slab steel, and the same quantity of concrete in the slab as in the Larkin Building, but the maximum stresses in the steel would be far less.

The unit stress in the slab rods at the middle of the long side belt of this straight flat slab is found by Equation (34) to be

$$f_s = \frac{WL_1}{175\, d_1\, A_1} = \frac{356\,700 \times 290}{175 \times 9 \times 22 \times 0.196} = 15\,230 \text{ lb. per sq. in.}$$

Similarly, the unit stress at the middle of the slab rods in the short side belt is

$$f_s = \frac{WL_2}{175\, d_1\, A_2} = \frac{356\,700 \times 240}{175 \times 9 \times 19 \times 0.196} = 14\,600 \text{ lb. per sq. in.}$$

By Equation (52) the unit stress in the diagonals at the center of the panel is

$$f_s = \frac{W\, L_1}{256\, j\, d_2\, A_2} = \frac{356\,700 \times 290}{256 \times 0.89 \times 8.56 \times 19 \times 0.196} = 14\,400 \text{ lb. per sq. in.}$$

It would require a load of more than 1 200 lb. per sq. ft. on such a mushroom slab to produce a maximum unit stress on the slab rods as great as was caused in the Larkin slab by the test load of 738 lb. per sq. ft. The difference is so great as to nullify entirely any such unwarranted claims as those quoted in the extract from the *Cement Era*.

It thus appears that a mushroom slab more than 65% stronger than the slab in the Larkin Building can be constructed at the comparatively slight additional expense of part of the steel required for the radial and ring rods of the mushroom head. How much more steel might be required for the mushroom is unknown, because no data are available as to the quantity of steel used in the top rods which were embedded in the top of the slab over the column heads, and in the frame supporting the slab rods in the column heads in the Larkin slab. In Mr. Lord's paper a photograph of the steel in place for pouring the concrete shows a rectangular frame support of this kind. Any claim for superiority of the Larkin structure with reference to strength and economy of materials over a properly designed "straight" flat slab is, in the writer's opinion, wholly without foundation.

The Donnelly Building.

The Donnelly Building, in Chicago, Ill., was designed and tested by Mr. A. R. Lord. Certain data of this test, as yet unpublished, have been put at the writer's disposal by C. A. P. Turner, M. Am. Soc. C. E. The panels in the slabs are 24 ft. 10 in. by 24 ft. 4 in. and 11 in. thick, with a drop of concrete at the head of each column, 9 ft. square and 9 in. thick, integral with the slab. The total thickness of the slab at the edge of the cap is 20 in.; the effective thickness from the center of the reinforcement in the head to the lower surface of the slab is 18 in.

Design load, 300 lb. per sq. ft.;
Weight of slab, 150 lb. per sq. ft.;
Test load, 750 lb. per sq. ft.;
Area of panel, 604.3 sq. ft.;
$L_1 = 298$ in.;
$L_2 = 292$ in.

All the reinforcement was of ½-in. round rods. Each long side belt had 26 rods, and each short side belt and diagonal belt, 25 rods. The computed maximum unit stress at the center of the short side belt, by Equation (34), is

$$f_s = \frac{292\ W}{175 \times 25 \times 0.196 \times 10.25}.$$

For a working load of $300 + 150 = 450$ lb. per sq. ft., $f_s = 9\,000$ lb. For the total test load of double this quantity, that is, 750 lb. per sq. ft., in addition to the weight of the slab itself, $f_s = 18\,000$ lb. when all the panels are equally loaded.

The deflection at the center of the panel may be computed for the test load of 750 lb. per sq. ft. by Equation (70) as follows, by assuming $\Sigma A = 6A_1$ as the reinforcement in the area over the head of the columns, this being 80% of that assumed in the standard mushroom design:

$$D_2 = \frac{604.3 \times 750 \times 292 \times 298^2}{10^{10} \times 25 \times 0.196}\left[\frac{1}{6.56 \times 9.75^2} + \frac{1}{10 \times 18^2}\right] = 0.384 \text{ in.}$$

The observed deflection was barely 0.3 in., which shows that the lines of inflection in this design were somewhat nearer the center of the panel than assumed in Equation (70), as they were also in the Larkin Building.

If a straight mushroom slab be designed with the same quantity of concrete in it as the Donnelly slab, the drop will furnish sufficient concrete to make the slab 1.2 in. thicker, giving a total thickness of 12.2 in. The computed stress at the middle of the side belts, in case all the slab belts remain unchanged, will be $f_s = 16\,100$ lb., instead of 18 000 lb. under the test load, and the computed deflection will be barely 0.4 in. at the center of the panel.

The Donnelly slab has a 9½-ft. square frame of 1⅛-in. round rods to support the slab rods in each head, which, with the lap, is more than 40 ft. long. A standard mushroom head would require more steel than this for radial and ring rods, but the quantity is very small compared with its considerable increase of strength, and this without increasing the deflection beyond a very conservative and in every way permissible value. Therefore, a straight flat slab design, such as the mushroom, appears to be preferable to one with a large drop like this. Means for making a rational comparison not having been available heretofore, it has not been possible to state positively that the unwarranted claims that have been made for designs with a large drop were unfounded; but that such is the fact is now demonstrable, as appears from the foregoing computations.

The St. Paul Bread Company Building.

The floor of the St. Paul Bread Company Building is a rough slab, 6 in. thick, and has 16 by 15-ft. panels, reinforced with ⅝-in. round rods, ten in each belt. The design load was 100 lb. per sq. ft. The slab, Fig. 19, was constructed in winter and frozen, but, as the final test was postponed until August, 1912, the slab was very fully cured, considerably more so, in fact, than most slabs when subjected to test. The test was made by Professor W. H. Kavanaugh, of the University of Minnesota, in the following manner:

First, extensometer measurements were made on seventeen 8-in. lengths of slab rods, which were exposed under a single loaded panel, three of these, Nos. 1, 2, and 3, being at the middle of three rods of one long side belt at the edge of the load, and the remaining fourteen distributed over the central area of one diagonal belt. Second, measurements of deflections were made at two points, one at the center of the panel and one near the middle of the interior edge of one long side belt. Third, the embedment of the rods was tested. Table 2 con-

tains the observed elongations due to change of loading at all the seventeen positions for each of the test loads, 108.4, 316.8, and 416.8 lb. per sq. ft., as well as after the removal of the load. A comparison of the observed elongations at symmetrical points reveals such discrepancies in the observations as to require some preliminary discussion.

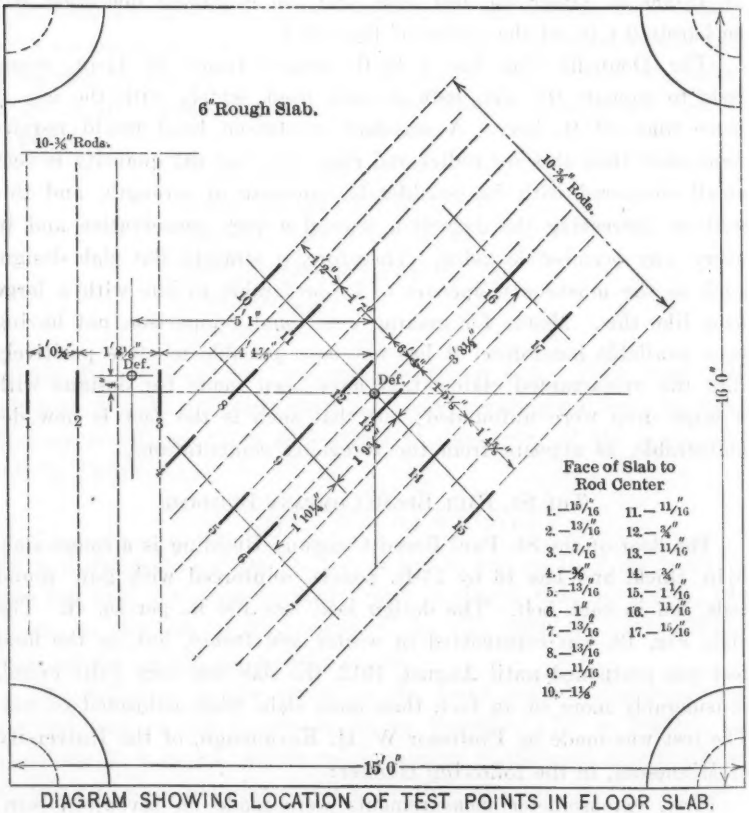


DIAGRAM SHOWING LOCATION OF TEST POINTS IN FLOOR SLAB.
SAINT PAUL BREAD COMPANY BUILDING.
"TURNER MUSHROOM SYSTEM."

FIG. 19.

Observations Nos. 5 and 6 were on a pair of diagonal rods on each side of and adjacent to the diagonal line of the panel, there being no rod exactly on the diagonal, and situated just beyond the edge of the other diagonal belt. No reason can be discerned for any difference between these elongations, but the wide

TABLE 2.—Observed Elongations, in Inches per Million Inches. Under Given Loads per Square Foot.

Note.—Obtain actual unit stresses by multiplying by 30.

Observation No.	Elongations under the Following Loads, in Pounds.			
	108.4	316.8	416.8	0
1	255	500	598	80
2	236	473	549	— 3
3	244	464	572	— 57
4	63	207	283	145
5	64	262	397	164
6	66	168	271	114
7	45	152	220	69
8	218	370	421	— 64
9	89	266	372	152
10	68	120	159	46
11	122	263	347	146
12	153	327	400	28
13	276	534	653	93
14	60	204	272	80
15	74	164	165	18
16	11	40	22	— 30
17	34	70	124	17

difference that appears must be due to some peculiarity in one of the rods, such as a crook or bend, or some lack of homogeneity in the concrete. Comparing Observations Nos. 5 and 6 with Nos. 4, 10, 15, and 16, which, being at about the same distance from the center, should, by Equation (49), have about the same elongations, it is found that No. 5 is abnormally large, and at the same time No. 16 is abnormally small. No. 8 is another set of abnormal results, which is evident from the fact that, being midway between Nos. 4 and 11, its elongations should lie between them; it is larger than it should be, with a final compression after removing the load. Nos. 7 and 17 should be the same, and Nos. 4 and 17 should be alike. The varying embedments of the portions of this rod which were observed show that there was probably a kink in it, which might account for the observed discrepancies. It is possible, however, that some such differences may appear when the loading is piled on one side of the panel before piling it on the other. No such explanation, however, will fit the case of Nos. 12 and 13, which are at the middle of the two rods adjacent to the diagonal line at the center of the panel. There appears to be no question that No. 13 is abnormally large, for No. 12 agrees well with others, being only a little larger than those at the nearby positions 9, 11, and 14, and the values at No. 13 are in wide disagreement with them. The

very considerable differences between results which should apparently be equal makes it evident how inexact single determinations must often be by reason of bends in the rods, lack of homogeneity in the concrete, etc., and emphasizes the importance of carefully laying belt rods straight and having them spaced uniformly, as well as embedded equally, before pouring the concrete, if consistent results are desired. It also shows the importance of checking all readings by readings at symmetrical positions.

It may be stated in general that the observed unit stresses and the deflections in this test are less than they would be for a slab tested at the stage of curing at which tests are usually made, a stage to which the equations apply more precisely. In consequence of this, all the computed results will exceed to some extent those actually observed.

Apply Equation (34) to compute the unit stress at Nos. 1, 2, and 3 of the long side belt. Assuming $d_1 = 5.3$ in.,

$$f_s = \frac{100\,000 \times 192}{175 \times 5.3 \times 1.1} = 18\,800 \text{ lb. per sq. in.}$$

in case of many panels equally loaded. The mean observed unit stresses for three rods at the edge of the load was 17 190 lb., the stress in each of the rods being practically the same, a fact that speaks well for the stiffness of the head. The computed unit stress at the center of the panel is

$$f_s = \frac{100\,000 \times 192}{256 \times 0.9 \times 5 \times 1.1} = 15\,150 \text{ lb. per sq. in.}$$

The observed unit stress at No. 12 was 12 000 lb., though at No. 13 the abnormal value of 19 600 lb. was found.

As the observations of stresses and deflections were made when only a single panel was loaded, and the computations assume that all the panels are equally loaded, any very close agreement of Table 2 with the observed results is not to be expected; nevertheless, comparison with that table shows that the computed results agree with the observations far better than the observations agree among themselves at such symmetrical points as admit of any comparisons.

The deflection at the center of the panel under a load of more than double the design load plus once the dead load, namely, 316.8 lb. per sq. ft., was 0.32 in., which is less than $\frac{1}{800}$ of the diagonal span. To

compute the deflections at the panel center, apply Equation (71), as follows:

$$D_2 = \frac{100\,000 \times 180 \times 192^2}{4.4 \times 10^{10} \times 5 \times 1.1} = 0.56 \text{ in.}$$

Computation of the deflection at the point near the middle of the interior edge of a long side belt gives a deflection for a load of 100 000 lb. on each panel of approximately 0.4 in.

TABLE 3.—DEFLECTIONS, IN INCHES, UNDER GIVEN LOADS PER SQUARE FOOT.

		Test Loads, in Pounds.			
		108.4	316.8	416.8	0
Center of panel	Observed	0.077	0.320	0.437	0.155
" " "	Computed	0.146	0.426	0.560	.....
Edge of side belt	Observed	0.065	0.247	0.332	0.124
" " " "	Computed	0.104	0.304	0.400	.....

As before stated, the somewhat large excess of computed over observed deflections is due to two circumstances: first, and principally, to the age and consequent stiffness of the slab; and secondly, at the edge, to the fact of a single panel load instead of many, neither of which circumstances is taken account of in the equations used.

THE STUDEBAKER AUTOMOBILE BUILDING.

The Studebaker Automobile Building, Michigan Avenue and 21st Street, Chicago, Ill., was erected in 1910, and is an example of a system of construction with two-way reinforcement only. The distances between column centers are 24 ft. and 23 ft. 9 in. Papers descriptive of this system may be found in the current technical journals.* All these papers, except the second, were written by T. L. Condron, M. Am. Soc. C. E., whose company designed the Studebaker Building. The last paper gives the results of tests of this and other buildings built on this system, while the second and third papers give the sizes and quantities of steel reinforcement in this particular building, from which it appears that the weight of steel reinforcement per panel was 2 600 lb.

Two kinds of floors are constructed on this two-way system; one is a perfectly flat slab of uniform thickness throughout; the other has

*Journal, Western Society of Engineers, December, 1909: *Engineering Record*, January 22d, 1910; *Engineering News*, January 27th, 1910; *Western Contractor*, March 12th, 1913.

a large so-called "panel" in the ceiling in the middle of the space between each four columns. The Studebaker Building is of this second kind, so that in the middle of each panel proper, 24 ft. by 23 ft. 9 in., there is this architectural "panel" about 18 ft. square. The thicker parts of the slab, extending from column head to column head, thus constitute shallow girders which in this case are 12 in. thick and about 6 ft. wide; and the central panels, 18 ft. square, are only 6 in. thick, giving a mean thickness of slab of 8.6 in., if the same quantity of concrete were uniformly distributed.

The design load was 100 lb. per sq. ft., and the dead load of the slab itself was also assumed at 100 lb. per sq. ft. The test load was applied on a single panel, and, in accordance with the Chicago Ordinances, amounted to 300 lb. per sq. ft., or a total of $300 \times 570 = 171\,000$ lb. This caused a deflection at the center of 0.34 in.

This floor will be compared with a mushroom slab, 8.6 in. thick, designed to carry the same loading.

The Studebaker slab had a thickness of 1 in. of concrete clear under the $\frac{3}{4}$-in. square girder rods, and $\frac{3}{4}$ in. clear of concrete under the $\frac{1}{2}$-in. round rods at the bottom of the central panel. The mushroom slab will be assumed to have twenty-five $\frac{3}{8}$-in. round rods in each belt, and 0.6 in. of concrete clear under the rods at the bottom of the slab, it being customary to make this fire-proofing layer less for smaller rods, and in any case not less than 0.5 in.

By Equation (34) the stress in the side belt under the dead load of 100 lb., plus the test load of 300 lb. per sq. ft., will be

$$f_s = \frac{228\,000 \times 288}{175 \times 7.8 \times 2.76} = 17\,400 \text{ lb. per sq. in.}$$

Mr. Condron states that he has used 17 000 lb. as the unit stress in the steel. Under a working load consisting of 100 lb. dead load plus 100 lb. design load the stress would be one-half the value computed above, namely, 8 700 lb. per sq. in.

The quantity of slab steel per panel in this design depends somewhat on the length of lap in the splicing. Without any lap, there would be in each 24 by 23.75-ft. panel:

Side rods,	25×24	= 600	ft.
Side rods,	25×23.75	= 594	"
Diagonal rods,	50×33.75	= 1 687.5	"
Total length of $\frac{3}{8}$-in. round rods		= 2 881.5	ft.

The weight of this is $2\,881.5 \times 376 = 1\,081.5$ lb. In round numbers, it will be assumed that the steel in the mushroom head, consisting of radial and ring rods, will bring the total weight up to 1 300 lb. per panel, which is just one-half that in the Studebaker Building with the same quantity of concrete.

Now, as to deflections: by Equation (71), the central deflection due to the test load is

$$D_2 = \frac{171\,000 \times (288)^3}{4.4 \times 10^{10} \times 2.76 \times (7.4375)^2} = 0.594 \text{ in.}$$

If the cross-section of the steel were doubled throughout, the deflection would be one-half of this, or 0.297 in., which is only slightly less than 0.34 in., which was the central deflection observed in the Studebaker test.

It will be noted that the formulas in the writer's investigations are dependent on the quantity of reinforcement, and not on the directions in which it runs, the only condition being that the belts shall cross in such immediate proximity to each other as to make their mutual action on each other effective. This does not occur with perfect uniformity everywhere in the two-way system under consideration, and, consequently, it would be expected to show somewhat greater deflections with the same quantity of steel than would the mushroom system.

The special claim made for this system by its designers is that in it the stresses in the steel reinforcement can be readily computed by the method of moments due to the loading. No tests have been published, so far as the writer knows, to substantiate this claim, and in his judgment such tests would show a difference, between extensometer observations and those obtained by the formulas published in the papers previously referred to, of not less than 50% of the observed amount, and probably more nearly 100 per cent.

This conclusion is based not only on the general theory of such structures, with which the writer has some familiarity, but on the computations just given in this paper, which fairly support this statement.

APPENDIX A.

Extracts from Report of Test of the Northwestern Glass Company Building.

By F. R. McMillan.

Description of the Building.—This is a four-story-and-basement warehouse, of the skeleton type of reinforced concrete construction, erected for the Northwestern Glass Company, of Minneapolis, from plans prepared by Bertrand and Chamberlin, Architects, and Mr. C. A. P. Turner, Consulting Engineer. In plan it is 66 by 163 ft., divided into 16 by 17-ft. panels. The floor is of the "Mushroom" type of flat slab construction, designed for a live load of 400 lb. per sq. ft. The 8-in. rough slab is reinforced with fifteen $\frac{3}{8}$-in. round rods in each direct and diagonal belt, as shown on Fig. 1. At the time of the test, the 2-in. cement finish which is to form the wearing surface was not in place.

Age and Conditions of Curing.—At the time of the test, the concrete in the test panels had been in place 5 months, but the weather had not been favorable for curing, except during the last few weeks. On the day the test panels were cast, December 6th, 1912, the mean temperature was 10°, with a maximum of 19° and a minimum of 0° Fahr. From this time until the end of March, 1913, the temperature practically remained below freezing. The usual precautions were taken to prevent freezing of the green concrete. Salamanders were kept burning in the basement until January 10th. The slab was kept covered with boards and canvas from the day it was poured until the erection of the forms for the second floor was begun. This interval was about a week or 10 days. Pouring of the second floor was begun on December 28th, and from this time until January 10th salamanders were kept fired under the second floor. The forms supporting the test panels were removed near the end of February. The last few weeks before the test the weather was very mild, and it is said that the concrete showed a noticeable increase in hardness during this period. At the time of the test, May 12th to 19th, the floor had all the appearance of an excellent hard concrete.

Plan and Description of the Test.—As the time available for the test, and the floor space that could be occupied in the basement, were limited by the necessity of caring for large shipments of glass, attention was confined to the measurement of deflections and deformations at critical points only. Very little choice was had in selecting points on the upper side of the slab at which to determine steel extensions;

for, with the type of extensometer used, measurements could only be taken on the top belt of rods and at points very near the surface.

Extensometer measurements were taken at various stages during the loading and unloading, and in all but one instance they were taken immeditely after each load was in place and again just before applying the next load. Deflections were taken at a number of intermediate stages at which no extensometer measurements were made. The extensometer measurements were made with the regular short-legged laboratory form of Berry strain gauge, reading over an 8-in. gauge line. The deflections were measured with a portable instrument specially designed to enable measurements to be taken at a large number of points without obstructing passage under the various panels.

The test load was of cement in bags, arranged in four piles in each panel, separated by aisles to prevent arching and to facilitate the taking of observations. The entire four panels were first loaded uniformly in three stages up to a total load of 540 lb. per sq. ft. of loaded area. The load from two panels was then nearly all transferred, in two stages, to the remaining two panels, making an intensity of 1 000 lb. and 750 lb. per sq. ft. on the loaded areas of these two panels. The removal of the load was made by stages in such a manner that the one panel retained its full load of 1 000 lb. per sq. ft. for 68 hours. The detail of areas and intensities of loading are shown on Figs. 5 and 6.

The accompanying tables and drawings show all the data of loading and the resulting steel stresses and deflections. These are self-explanatory. The steel stresses have been calculated from the extensometer measurements on a basis of 30 000 000 lb. per sq. in. for the modulus of elasticity. Fig. 13 shows the actual thickness of the test slabs and the depth of the rods at the top on which extensometer measurements were made.

TABLE 4.—TIME SCHEDULE OF LOADING.

TEST OF NORTHWESTERN GLASS COMPANY BUILDING.

Date, 1913.	Load No.	APPLYING LOAD.			INTERVAL, LOAD IN PLACE.	
		Began.	Noon.	Finished.	Hr.	Min.
May 13.....	1	10:15 A. M.	12-12:30	1:10 P. M.	.....	.....
May 14.....	2	9:30 A. M.	12-12:30	2:45 P. M.	25	35
May 15.....	3	10:30 A. M.	12-12:30	1:50 P. M.	23	5
May 16.....	4	12:45 A. M.	.....	3:00 P. M.	25	10
May 17.....	4a	11:00 A. M.	12-12:30	3:00 P. M.	24	0
May 17.....	5	3:45 P. M.	.....	4:45 P. M.	1	45
May 18.....	6	9:45 A. M.	.....	10:40 A. M.	17	55
May 18.....	7	11:15 A. M.	.....	12:00 NOON.	1	20
May 19.....	8	10:30 A. M.	12-12:30	12:53 P. M.	24	53
May 19.....	9	1:30 P. M.	.....	4:00 P. M.	3	7

TABLE 5.—Summary of Loading.

Load No.	Total Panel Loads.				Load per Square Foot Over Loaded Area.			
	A	B	C	D	A	B	C	D
1	31 540	31 445	29 355	29 070	181	178	169	162
2	68 780	67 830	68 875	66 025	395	385	396	364
3	96 045	93 955	97 090	92 910	552	533	558	512
4	59 470	130 815	58 995	130 720	342	742	339	720
4a	59 470	130 815	6 745	182 970	Same.	Same.	39	1 004
5	None.	132 430	Same.	Same.	None.	751	Same.	Same.
6	"	60 135	"	"	"	341	"	"
7	"	None.	"	"	"	None.	"	"
8	"	"	"	94 715	"	"	"	521
9	"	7 000	7 000	None.	"	40	40	None.

TABLE 6.—Loading Data, Panel A.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 174.1 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	A-1.	A-2.	A-3.	A-4.		A-1.	A-2.	A-3.	A-4.	Loaded area.	Whole panel.
1	8 265	9 025	7 315	6 935	31 540	182	192	178	172	181	116
2	17 385	18 525	16 815	16 055	68 780	382	394	408	397	395	253
3	24 130	25 270	23 845	22 800	96 045	531	537	579	565	552	353
4	15 105	16 150	14 440	13 775	59 470	332	344	350	341	342	219
4a	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.
5	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	"	"	"	"	"	"	"	"	"	"	"
9	"	"	"	"	"	"	"	"	"	"	"

TABLE 7.—Loading Data, Panel B.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 176.3 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	B-1.	B-2.	B-3.	B-4.		B-1.	B-2.	B-3.	B-4.	Loaded area.	Whole panel.
1	7 600	7 980	8 550	7 315	31 455	169	191	187	167	178	116
2	15 960	17 100	17 955	16 815	67 830	355	409	393	384	385	249
3	22 135	23 845	24 415	23 560	93 955	492	571	535	538	533	345
4	30 875	33 345	33 535	33 060	130 815	686	798	734	755	742	481
4a	Same.	Same.	Same.	Same.	130 815	Same.	Same.	Same.	Same.	Same.	Same
5	32 490	"	"	"	132 430	722	"	"	"	751	487
6	15 485	14 820	15 390	14 440	60 135	344	355	337	330	341	221
7	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None
8	"	"	"	"	"	"	"	"	"	"	"
9	"	3 500	3 500	"	7 000	"	84	77	"	40	26

TABLE 8.—Loading Data, Panel C.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 173.9 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	C-1.	C-2.	C-3.	C-4.		C-1.	C-2.	C-3.	C-4.	Loaded area.	Whole panel.
1	7 885	7 220	6 080	8 170	29 355	157	172	163	183	169	108
2	19 285	16 340	14 820	18 430	68 875	383	390	398	414	396	253
3	27 360	23 085	20 615	26 030	97 090	542	551	554	586	558	357
4	16 435	14 060	12 635	15 865	58 995	326	336	340	356	339	217
4a	4 750	None.	None.	1 995	6 745	94	None.	None.	45	39	25
5	Same.	"	"	Same.	Same.	Same.	"	"	Same.	Same.	Same.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	"	"	"	"	"	"	"	"	"	"	"
9	None.	3 500	3 500	None.	7 000	None.	83	94	None.	40	26

TABLE 9.—Loading Data, Panel D.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 181.6 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	D-1.	D-2.	D-3.	D-4.		D-1.	D-2.	D-3.	D-4.	Loaded area.	Whole panel.
1	7 410	7 315	6 840	7 505	29 070	168	158	159	156	162	107
2	16 150	16 435	16 055	17 385	66 025	365	364	372	362	364	243
3	22 895	22 895	22 515	24 605	92 910	520	494	522	514	512	342
4	32 680	31 825	31 350	34 865	130 720	740	686	728	727	720	480
4a	47 405	45 980	38 760	50 825	182 970	1 073	994	900	1 059	1 004	672
5	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	22 895	22 895	22 515	26 400	94 705	518	494	522	550	521	348
9	None.	None.	None.	None.	None.	None.	None	None.	None.	None.	None.

TABLE 10.—TEST OF NORTHWESTERN GLASS COMPANY BUILDING.

SLAB DEFLECTIONS. PANELS B AND A.

Deflections are given in thousandths of an inch.

Load No.	Date, May, 1913.	DEFLECTIONS AT POINT NUMBER.											
		1	2	3	28	29	30	4	5	6	7	8	9
Zero.	12th	0	0	0	....	....	....	0	0	0	0	0	0
1	13th	11	22	16	....	....	....	11	15	43	32	34	18
1	14th	19	23	14	....	....	....	11	15	43	28	30	12
2	14th	59	102	60	....	....	....	44	66	105	115	121	56
2	15th	77	121	70	0	0	0	54	76	118	130	141	70
3	15th	130	206	111	86	101	69	89	124	182	219	252	135
3	16th	144	237	131	105	145	98	101	144	211	260	282	147
4	16th	158	260	138	156	251	178	94	131	186	225	241	120
4	17th	169	275	146	173	281	208	92	132	188	226	241	122
5	17th	154	250	132	170	293	211	77	102	141	160	149	71
5	18th	153	248	131	179	310	226	71	96	136	151	140	67
6	18th	....	....	....	103	187	143	....	....	....	....	....	....
7	18th	97	148	78	29	63	50	60	83	121	128	122	56
7	19th	83	139	70	20	55	40	60	82	120	129	127	49
8	19th	....	138	....	19	51	40	....	76	115	121	125	*
9	19th	89	142	74	22	62	*	50	69	105	115	116	....
9	20th	91	145	74	26	54	....	53	69	106	115	119	....
9	24th	82	133	77	17	37	....	47	63	98	104	107	....
10	June 5th	*	128	78	*	*	....	46	64	*	105	108	....

*These points were inaccessible. Note that deflections given for Points 28, 29, and 30 represent deflections below the position of the slab at Load 2. At all other points the deflections shown are from the initial position of the slab.

TABLE 11.—TEST OF NORTHWESTERN GLASS COMPANY BUILDING.

SLAB DEFLECTIONS. PANEL D.

Deflections are given in thousandths of an inch.

Load No.	Date, May, 1913.	DEFLECTIONS AT POINT NUMBER.												
		10	11	12	13	14	15	16	17	18	19	20	21	27
Zero	12th	0	0	0	0	0	0	0	0	0	0	0	0	0
1	13th	15	25	25	32	10	17	32	37	37	27	22	11	0
1	14th	10	19	23	28	6	9	22	32	31	21	16	4	10
2	14th	49	78	104	105	42	55	79	106	109	48	78	33	47
2	15th	52	88	92	111	58	73	99	118	131	72	87	38	55
3	15th	74	124	154	166	72	95	136	168	200	95	89	50	77
3	16th	86	140	170	187	80	106	149	189	191	111	132	54	81
4	16th	92	143	179	235	94	128	183	242	251	147	137	80	89
4	17th	92	148	173	247	101	142	199	264	276	161	147	90	90
4*b*	17th	.....	.....	.....	.....	.....	.....	.....	.....	333	.....	....	....	....
4*a*	17th	.....	.....	.....	.....	.....	.....	.....	.....	396	.....	....	....	....
5	17th	100	164	193	343	140	198	276	387	416	250	172	150	110
5	18th	99	169	208	374	150	214	297	423	456	278	182	160	113
6	18th	.....	172	211	387	154	215	308	434	466	282	188	160	116
7	18th	105	176	213	392	159	223	309	440	472	283	191	166	120
7	19th	109	185	217	415	168	237	332	474	510	304	202	174	124
8	19th	89	145	175	330	133	184	257	366	395	234	156	129	102
9	19th	57	95	101	188	76	106	149	204	216	123	93	67	61
9	20th	60	91	93	178	80	103	140	195	208	113	91	68	61
9	24th	56	83	96	147	.....	94	134	176	187	101	83	61	59
10	June 5th	51	*	*	*	*	*	118	157	169	93	*	*	*

*These points were inaccessible. Note that the deflections given for Point 27 represent deflections from the position of the slab under Load 1. At all other points the deflections shown are from the initial position of the slab.

TABLE 12.—TEST OF NORTHWESTERN GLASS COMPANY BUILDING.
STEEL STRESSES—UPPER SIDE OF SLAB.
Stresses are given in thousands of pounds per square inch.

Load No.	Date, May, 1913.	STRESSES AT GAUGE LINE NUMBER.										
		101	102	103	104	105	106	107	108	109	110	111
Zero	13th	0	0	0	0	0	0	0	0	0	0	0
1	13th	0.4	2.2	2.8	0.4	0.8	1.7	2.0	2.8	1.3	0.2	2.4
2	14th	1.5	5.8	7.1	2.0	2.7	9.7	9.3	11.6	6.4	2.0	12.0
2	15th	2.0	5.8	7.0	2.1	2.7	9.9	9.8	12.1	6.4	2.0	13.6
3	15th	2.8	7.9	9.7	2.8	2.4	14.8	15.8	18.7	10.1	2.8	20.2
3	16th	2.7	8.7	9.7	2.9	2.6	15.5	17.5	20.0	10.6	2.8	22.4
4	16th	3.0	11.2	11.2	4.5	3.7	13.8	15.5	17.8	9.8	2.1	19.6
4	17th	3.1	12.1	13.6	4.9	3.6	14.3	15.3	18.0	10.2	2.7	19.3
5	17th	3.4	16.3	18.3	7.4	5.2	10.9	10.6	13.0	9.4	2.1	14.2
5	18th	4.0	17.3	18.3	8.3	5.9	12.1	10.3	13.9	10.0	2.8	14.4
7	18th	4.2	18.1	19.3	8.5	5.8	11.4	7.6	11.9	9.8	2.8	14.4
7	19th	5.2	19.3	19.8	9.7	7.3	12.0	8.7	12.1	10.1	3.4	14.7
9	19th	3.6	5.8	6.8	4.4	5.6	8.2	7.7	8.1	5.1	0.9	11.6
9	20th	3.3	4.7	5.6	3.5	4.3	7.1	7.0	7.6	4.2	0.1	10.3

All stresses in this table are tensions.

TABLE 13.—TEST OF NORTHWESTERN GLASS COMPANY BUILDING.
STEEL STRESSES—UNDER SIDE OF SLAB.
Stresses are given in thousands of pounds per square inch.

Load No.	Date, May, 1913.	STRESSES AT GAUGE LINE NUMBER.										
		1	2	3	4	5	6	7	8	9	10	11
Zero	12th	0	0	0	0	0	0	0	0	0	0	0
1	13th	1.0	0.2	1.5	0.8	1.1	2.0	0.4	0.5	1.0	—0.4	1.5
2	14th	2.0	1.7	3.0	2.3	3.7	4.0	3.3	3.6	4.9	4.9	4.0
2	15th	1.8	1.7	2.7	1.2	2.7	4.6	2.8	3.4	4.7	5.2	4.0
3	15th	2.2	3.0	4.3	2.8	4.0	6.1	5.7	6.7	8.1	12.6	9.9
3	16th	2.5	3.4	4.6	2.8	4.3	6.4	6.5	7.5	8.2	15.0	12.1
4	16th	3.4	5.3	7.7	4.0	6.1	7.9	7.0	7.2	6.8	16.3	17.8
4	17th	3.9	5.8	8.4	4.8	6.9	8.8	7.5	7.1	7.5	17.0	19.0
5	17th	6.0	10.9	14.2	8.6	11.7	11.8	10.0	9.0	7.6	14.9	19.2
5	18th	7.1	12.2	15.7	10.3	14.2	13.5	10.9	8.8	6.9	15.1	20.5
7	18th	6.5	11.8	15.9	10.8	14.7	13.9	11.6	9.4	8.4	7.5	9.4
7	19th	7.1	12.5	17.1	11.9	16.6	15.3	12.7	10.7	8.2	6.7	9.0
9	19th	3.0	4.8	6.8	4.9	8.0	6.2	6.0	5.4	5.8	7.9	10.7
9	20th	3.1	4.4	5.9	4.4	7.1	6.1	5.5	4.3	4.5	7.9	9.6

The minus sign indicates compression. Where no sign is used the stress is a tension.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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MODERN PIER CONSTRUCTION IN NEW YORK HARBOR.

Discussion.*

By Charles W. Staniford, M. Am. Soc. C. E.†

Mr. Staniford.

Charles W. Staniford, M. Am. Soc. C. E. (by letter).—Some of the discussion has wandered into questions not germane to the subject matter of the paper, and has entered into matters pertaining to harbor design and general methods of construction.

The particular object of the paper was simply to show what had been accomplished by the Department of Docks in producing something more permanent than the ordinary New York wooden pier, and at about the same cost.

It is the writer's opinion, however, that all the discussions are valuable, as they bring out many admirable points in connection with harbor design and general designs for actual construction.

It is repeated that the old wooden pier, as built in New York Harbor up to 10 years ago, because it was easily removed and reconstructed, was one of the greatest factors in developing the harbor facilities. Instead of the shipping being compelled to adapt itself to the location and operation of massive permanent structures located along the water-front, as in English and Continental harbors, where there are stone quays and piers, the structures for berthing vessels and taking care of cargoes in New York Harbor were located to meet the conditions of shipping as these conditions developed and increased facilities were required. As the size of the vessels increased, the docks grew larger, in width as well as length, in proportion to the width of the slips, that is, the distance between piers increased to accommodate the greater beam of ocean steamers. It can readily be

* Continued from November, 1913, *Proceedings*.

† Author's closure.

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seen that if the New York water-front was lined with a system of massive and permanent stone, steel, or concrete structures, the development of the harbor could only proceed at enormous expense, and consequently with much less rapidity than is made possible by building structures of lighter form. The type formerly built by the Department met for the time all requirements, but, at little added expense, many objectionable features of the former construction have been eliminated, thus doing away with constant repairs; and, for the present, at least, the Department has a type which is durable and at the same time inexpensive, and admirably meets the present-day demands of shipping. The result of this more modern method of construction has been shown in the paper, and the figures substantiating the statements are based on fact, not conjecture.

Mr. Harris questions the lasting qualities of the asphalt wearing surface, and infers that a 3-in. deck sheathing would outlast the asphalt. The experience of the Department with deck sheathing is absolutely as cited in the paper, and not a matter of inference. Repairs to New York piers are absolutely needed at short intervals, resulting in frequent renewal of the whole deck sheathing, the wear on the latter being the result of a traffic movement probably unequaled in the world. In fact, this whole paper deals with conditions of this kind, where structures are under intense use constantly, and not with docks built in Navy Yards or in any place for public function, where team traffic is almost unknown, or where the docks are practically intended as a berthing place for steamers while undergoing repairs.

As a concrete illustration of the inexpensiveness of the asphalt wearing surface, the piers of the Chelsea Section may be cited. On these nine piers, each approximately 800 ft. long and 125 ft. wide, asphalt wearing surfaces were laid, and for a period of 5 years, under the most extreme traffic conditions, no repairs were necessary. Under the contract for this work, 96 989 sq. yd. were laid, and a bond for 5 years was given to maintain the asphalt wearing surface; at the expiration of this 5-year bond period, repairs were only needed on 780 sq. yd., costing $800, or less than 1% of the total cost. From the expiration of the bond period to the present day no repairs of these surfaces have been necessary; and, furthermore, no repairs are needed at the present time. Fourteen other piers may be cited where the same result has been secured by the use of asphalt.

As stated in the paper, for light traffic, this asphalt wearing surface may be eliminated, but, for heavy traffic, it has been found absolutely necessary to put some deck wearing surface over the concrete; and in some other localities the writer has advocated wood block or brick pavement for this purpose. Granite block pavement has been considered, but was not adopted because it would add so much to the weight.

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Mr. Harris questions the facility of making repairs to decks of this kind. These repairs, like all others relating to dock and harbor structures, must be made by specialists in work of this class. The Department force, consisting of twenty pile-drivers with their crews, is constantly engaged in doing just such work.

TABLE 4.—PIERS ON WHICH ASPHALT SURFACES HAVE BEEN ADOPTED.

Location.	Area, in square yards.	Price per square yard.	Elapsed time, in months.	Area repaired, in square yards.	Percentage of area, in stated time.	Percentage of area repaired per year.
Piers 53 to 62, N. R.	96 989	$0.90	34	40	.....	.....
		.....	3	70	.....	.....
		.....	3	175	.....	.....
		.....	13	280	.....	.....
		.....	2	145	.....	.....
		.....	5	70	.....	.....
		Total:	60	780	0.8	0.16
Pier, W. 40th Street, N. R.	2 817	$1.48	48	none.	.....	.....
" 5, E. R.	4 315	1.19	30	none.	.....	.....
" 6, E. R.	3 043	1.19	12	25	.....	.....
			9	50	.....	.....
		Total:	21	75	2.5	1.43
Pier 22, E. R.	2 785	$1.48	60	20	0.7	0.14
" E. 5th Street, E. R.	2 876	1.18	22	25	1.0	0.55
" E. 20th Street, E. R.	3 602	1.49	46	70	2.0	0.52
" E. 95th Street, E. R.	1 614	0.90	27	none.	.....	.....
" E. 100th Street, E. R.	1 095	0.90	28	none.	.....	.....
" E. 110th Street, H. R.	1 964	1.49	48	none.	.....	.....
" Canal Street, Stapleton.	2 296	1.23	60	20	1.0	0.20
Platform, Stapleton Ferry	1 594	2.40	60	35	2.2	0.22
Pier, 52d Street, Brooklyn	2 719	0.84	23	25	1.0	0.52
" Whale Creek, Brooklyn	4 788	0.98	16	none.	.....	.....
" Jamaica Avenue, Queens.	742	1.45	18	none.	.....	.....

It is mentioned that the eventual destruction of caps and stringers by continuous repairs deserves consideration. This is just what was considered in the design of piers of this style. By eliminating all the stringers and all the deck, leaving practically little more than the timber caps, the quantity of timber to be renewed is reduced to an absolute minimum, the life of the caps, in addition being prolonged considerably by the use of the new system with its water-proof deck. In time, when repairs to this composite structure are required, owing to the rot of this insignificant quantity of timber underneath, such repairs will not necessitate the disturbance of the concrete deck, as all the work may be done below it by simply supporting the structure between alternate pile rows, removing the caps, and bench-capping

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the piles, the usual method by which similar repairs have been made by the Department for 25 years or more.

The repair of an occasional pile in the interior of the pier, on account of rot, is accomplished without any trouble by cutting the pile and replacing the head by bench-capping or splicing, thus making the structure as strong as the original. This has been done in a number of instances in New York, even under elaborate buildings, such as ferry houses, without any trouble whatever, and without any interruption of traffic.

The design of this pier offers relief from constant repairs, and makes unnecessary the expense of actual reconstruction from mean low water up, for the experience with the asphalt deck surface has covered a sufficient time to prove that repairs have been greatly eliminated by this feature, and the structure is actually preserved by the asphalt deck surface.

The writer gave what he considered to be full recognition to the design of the piers being built in the Brooklyn Navy Yard, as being a good type for producing an absolutely permanent structure in these waters. To produce anything in the line of permanent development, however, is simply a question of expense. Mr. Harris has given some figures relative to the cost of piers in South Brooklyn, and has inferred that these prices were not a fair criterion, it being contended that the piers were built at extremely low cost, and with no profit to the contractor. This contention is difficult to prove. In the paper, the cost data for these piers are correct, and are repeated and elaborated as follows: The 31st Street Pier was let to the lowest bidder at $174 587. This was one of twelve bids, the next four higher being: $183 820, $192 462, $195 897, and $196 725. The 33d Street Pier was let to the lowest bidder at $236 500. This was one of seven bids, the next three higher being: $244 587, $246 332, and $254 897.

In each case the bidders for these piers were among the most reliable dock-building contractors in New York; and without question, any one of them could have performed the work successfully for the amount of his bid.

The New York Department of Docks has developed many designs for absolutely permanent structures, not only on the lines of the piers in the Brooklyn Navy Yard, made permanent from low water up, but absolutely permanent structures with concrete piles, and various forms of filled-in piers, some of which are without doubt as good as those cited. In any event, no two designs are exactly the same. Where the money is available, and the cost of deck repairs will necessarily be slight, and where a permanent structure is warranted, Mr. Harris is unquestionably right in suggesting that the structure should be made permanent from low water up.

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Mr. Harris states that the generally accepted opinion that there are no marine borers in New York Harbor is in error. He states that they are found in the Lower Harbor, and that perhaps under certain conditions they may develop with wind and tide on the South Brooklyn shore. The writer fails to see why he cites the fact that he had found sea borers at Fort Lafayette. This question of the presence of the teredo, limnoria, or other wood-boring animals was settled by the Department more than 40 years ago. In fact, it was not only settled by the Department, but by all the corporate interests which have helped to build up New York Harbor while spending several hundred millions of dollars in building wooden structures. Unquestionably, if New York had remained a settlement with none of the evils attendant on the growth of a city of even ordinary size, these wood borers would have remained in the harbor, and the present form of construction would not have obtained. This has not been the fact, however, and 30 years ago the last evidences of the activity of these animals disappeared in all pier construction built either by the City or by others in the Upper Harbor.

In presenting this pier as a type of construction for New York Harbor, the writer took it for granted that it would be understood as applying to piers to be built in the harbor proper, and not for outlying sections nearer the ocean, where, of course, the teredo exists, this form of sea life requiring clear and fresh salt water in order to live. These wood borers have long been driven from the harbor proper. Even 10 years ago, the Department, in approaching the zone of danger (at the end of the Lower Bay) adopted the plan of creosoting all piles, but even in such localities, namely, along the northerly shores of Staten Island, at St. George, and other places where the teredo had been somewhat active, untreated piles which had been in the water for 8 years have been pulled and have shown absolutely no sign of any wood borers.

Before determining to use timber in the South Brooklyn Section, the Department for a number of years made a most exhaustive investigation, with divers, of hundreds of existing piers, bulkheads, and other structures along the water-front, looking for signs of infection by the teredo. The result showed that, although this sea worm had been somewhat active there in years past, it is now practically extinct.

Furthermore, for years past, the Department has been continually repairing the older structures built by the City. In the course of these repairs, the renewal of piles, even in the older structures, is found necessary only at rare intervals in the interior of the piers where they have been protected by the timber deck; though near the sides, where exposure is greater, more extensive repairs are needed. Also, the Department is continually removing old structures, some of them the oldest in New York Harbor, to make way for improvements. In these

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removals every pile is examined, under orders, for any evidence of the teredo, in order to obtain some idea of the time when it disappeared from that particular locality. In some instances the piles show that they were attacked by the teredo in the olden days, but, probably on account of increasing pollution of the water, it had ceased to exist. Piles removed from structures built within the past 25 years are as good as the day they were driven, and are constantly used again in other work.

Recently, the Department has been removing some of the old Brooklyn ferry houses on the East River, these structures being among the very oldest on the water-front. In pulling the piles, each was examined with particular care to see if the teredo had made any ravages, but no evidence was found in any case.

The following is a concrete example of the wonderful preservation of these piles and the non-existence of the teredo. When the old Pennsylvania Ferry House, at the foot of Cortlandt Street, one of the oldest structures along the lower water-front near the Battery, was being removed, the writer visited the premises in order to determine whether it would be advisable to use some of the old piles, which happened to be in the right position to take their place as part of the grillage for the concrete retaining wall which was to run under the new ferry house. He found that the piles, cut several feet above low water, were in a remarkably good state of preservation; the foreman, on being told that the piles were as good as the day they were placed, replied, "They are better than the day they were driven." They were subsequently retained and remained in the permanent structure as specimens of absolutely perfect piles, thus saving the cost of placing a number of new ones.

Mr. Snow says:

"There are many examples of concrete construction in sea water in Boston Harbor, and in every instance known to the speaker there has been serious disintegration between high and low tides."

In New York this condition does not obtain in the concrete works produced by the Department of Docks. A few other concrete bulkheads have disintegrated in this way, notably at the Brooklyn Navy Yard, where such a bulkhead, approximately a mile in length, has entirely disintegrated and been removed. The last piece of this concrete wall is now being demolished near the end of the Cob Dock, where absolute disintegration has taken place throughout, extending many feet back from the face, including the front piles, ends of caps, etc.

It is presumed that the Government will replace this wall with a new concrete structure, and then all the old Navy Yard concrete wall will have been reconstructed.

Up to the present time there has been no failure in any part of more than 8½ miles of concrete and granite wall construction, built

by the Department of Docks and Ferries, during a period of more than 40 years. Mr. Staniford.

Frank W. Hodgdon, M. Am. Soc. C. E., Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners, writes as follows:*

"Practically all the structures built of concrete in Boston Harbor have been badly disintegrated between high and low-water marks."

To this the writer replied as follows:†

"That it will be of interest to your readers is evidenced by the fact that this office is in continual receipt of letters of inquiry from all parts of the world, not only from harbor engineers, but from engineers engaged in the design of bridges in order to ascertain the condition of such work built of concrete in salt water, in view of the failure of such structures throughout the world.

"It certainly is a question concerning which engineers should receive all information possible whereby people at large may receive due consideration in assuming the financial responsibility in future for constructions of this kind which are supposed to be built with a view to permanency.

"I will not attempt to give the history of anything but city construction work in New York Harbor, and the condition of same at the present time. Most of the work done here during the past forty years has been done by the Department of Docks, or what is now known as the Department of Docks and Ferries of the City of New York. During this period the city has been engaged in the construction of a permanent sea-wall, located principally around lower Manhattan, below 80th Street. This has been added to recently by quite extensive construction at North Brother Island, in the Borough of the Bronx, at Whale Creek in the Borough of Brooklyn, and at the Gowanus Section, South Brooklyn.

"This sea-wall in a general way varies in type of construction, according to the character of foundation, but the essential features above the foundation are very similar in all types. Almost every type of bottom is encountered in New York Harbor, while the wall has been built to meet these varying conditions, about twenty-five different modifications having been used thus far in the foundation work to meet the conditions imposed. There are three general types, however; one built upon solid rock, one built upon hard bottom, such as good, stiff clay or compact sand, and one built on soft river mud, forming a wall supported by mud flotation.

"In many parts of the waterfront of Manhattan, for instance, bedrock is over 200 ft. below the river bottom, overlaid with soft mud. In the rock sections the wall has acquired a height above the rock as much as 50 ft. In other rock sections the wall is founded upon rock that has had to be blasted in order to create a sufficient depth for wharfage in front, and to take off sufficient rock for a stepped-off bottom in order to safely found the wall against the slope of the natural rock.

* *Engineering Record*, June 10th, 1911, p. 655.
† *Engineering Record*, August 5th, 1911, p. 176.

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"In the soft mud bottoms the longest piles are used. In most types requiring pile foundations, the piles are usually cut off at a depth of about 15 ft., below mean low water, and from the top of these piles the wall proper is built.

"In the case of the rock bottom type the wall proper may be considered to commence at the top of the bag foundation which is built immediately on the rock bottom to a proper grade, for receiving the wall proper. From the tops of these pile and concrete bag foundations, the wall is built up to the vicinity of mean low water of concrete blocks, varying according to design from about 20 to about 95 tons in weight. These concrete blocks are moulded in air in mould boxes and are shipped on scows from the yard at which they are fabricated to any point on the waterfront where they are to be used. For handling these blocks we use two derricks, according to weight of block, a 40-ton derrick and a 100-ton derrick.

"Above these blocks the type of construction is generally the same, consisting of four courses of granite facing, of headers and stretchers, dove-tailed into a thick backing of concrete deposited en masse in the rear of the granite, and all carried up to the grade for the granite coping. In several places concrete deposited in situ forming a homogeneous mass with the concrete backing is substituted for granite and carried up to the coping course. This is usually done at points particularly where piers project from the bulkhead, as this portion of the wall is covered by the pier and, therefore, not visible. This type of construction with concrete face, instead of granite face, appears also in the general construction of the Claremont Section, the 116th Street Section, the Gowanus Section in South Brooklyn, the Whale Creek Section in Brooklyn, the Fulton Section on the lower East River, and part of the Old Slip Section on the East River.

"All of this wall construction has been built in the river itself, generally well off from the shore, in salt water, in a tide with an average range of 5 ft., varying in extremes to 8 ft. The tidal and current conditions in most parts of the East River waterfront are so severe that a diver is not able to stand up and work for more than half an hour in slack water, necessitating the construction of temporary breakwater barriers as a protection for the diver and for work against the current.

"The Department of Docks and Ferries, up to the present time, has built about 8¼ miles of this river or sea-wall, requiring in its construction in the neighborhood of 6 000 concrete blocks. In the course of years the Department has had many opportunities for examining the condition of this wall in connection with the building of sewers, etc. Up to the present time no disintegration has been discovered that can be attributed to the existence of the structure in salt water. The concrete itself is in an admirable state of preservation, absolutely hard, and is undergoing no regular process of disintegration. In some cases between high and low water marks, where the concrete has been deposited in situ, there are indications of ice abrasions, but nothing whatever to cause any softness or to expect ultimate trouble.

"In some cases there are indications of the effect of wave action or scour on the freshly deposited concrete within the tidal range, due

CONCRETE BLOCKS BUILT AND PLACED IN WALL BY DEPARTMENT OF DOCKS, AND REMOVED BY THE EDISON COMPANY IN THE CONSTRUCTION OF INTAKE TUNNELS.

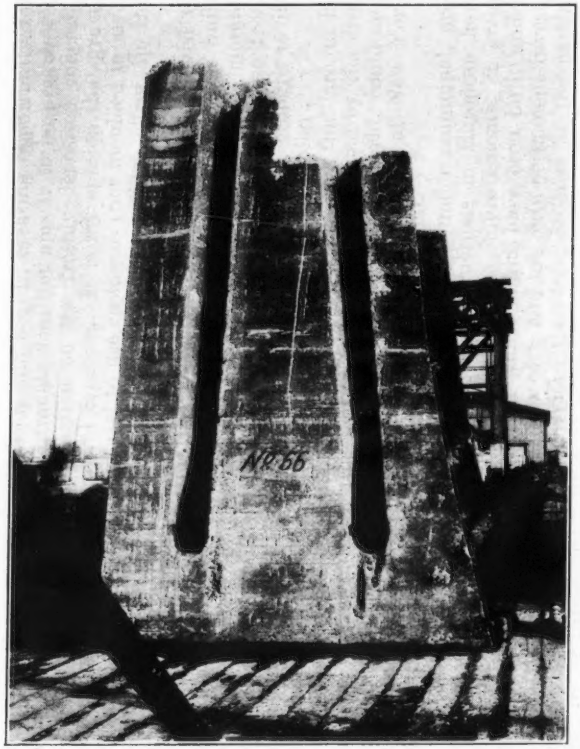

FIG. 13.—END VIEW.

FIG. 14.—BACK VIEW.

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to carelessness in work on the mould boards, creating opportunity for tidal or wave scour, but this trouble does not appear to be progressive after the original scouring action on the freshly deposited concrete, excepting in the case of gravel concrete. It is my opinion that gravel concrete is not a proper material to use en masse (not moulded in air), where it is to be subjected to wave action or tidal effect in the process of depositing.

"Up to the present time the Department has not been obliged to spend much money on repairs to the sea-wall. This wall was built to be permanent, and at the present day all of the exposed concrete, which in the course of a few years may become a little worse from abrasion than at the present time, can be repaired and brought to its original state with the expenditure of a few hundred dollars.

"The extreme conditions that exist in Boston and that are to be seen in certain parts of this harbor, outside of work done by the city, and, in fact, in all of the various harbors of the world, could not exist and continue in the municipal work of the City of New York under its present financial control and government.

"Concrete harbor works built in other cities under municipal, private or corporate control, as well as constructions in European harbors, were, of course, all built originally as improvements, at great expense, over timber construction which might have been built under ordinary expenditure at a far less cost, and are only permanent investments, according to their behavior; therefore, because of the partial failure or threatened failure of so much concrete work in harbor construction, leading engineers throughout the world are assuming unusually expensive construction in their endeavors to produce permanency, by going to the other extreme and using granite or stone.

"Notwithstanding the failure or partial failure of concrete in the presence of salt water, in certain localities, the fact remains that this sea-wall which has been under construction by the City of New York for 41 years, is at the present time an excellent piece of work and is subject to the same climatic conditions as all cities on the Northern Atlantic Coast with the attendant ice, cold and rain characteristic of this latitude."

With reference to the concrete blocks mentioned by Mr. Betts as having been removed from the bulkhead wall after immersion in the salt water of the harbor for 10 years, Figs. 13 and 14 are submitted. These are from photographs of the blocks after they had been removed from the wall at 38th Street, East River, and placed on the Department Dock at the East 24th Street Yard. These blocks, weighing about 80 tons each, and recorded as Nos. 66 and 82, respectively, were fabricated on June 20th and August 16th 1902, and were set in the East 38th Street bulkhead wall on July 14th and August 30th, 1902. They were removed from the wall by the Edison Company, in the construction of intake tunnels, on October 31st, 1912.

The abrasions along the top and parts of the bottom edges of these blocks are the result of chiseling, in taking the blocks out of the wall and breaking their contact with the concrete backing and

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granite which surmounted them. They are in a perfect state of preservation, showing no disintegration, even to the extent of the displacement of a particle. The edges are practically knife-edges, and the sharp-cut ridges along the joint lines of the mould boards show clearly. These blocks will be used in other work of the Dock Department when convenient.

Mr. Taft has presented a design for a permanent concrete dock which no doubt would answer for some localities where the bottom would permit of constructing a foundation such as he recommends. It is novel in many of its features, and has many good points; but the question of decreasing the thickness of slab in the type of deck now being built by this Department would not be considered here. This is a question that cannot be decided analytically, but must be answered by experience with structures subject to the heavy side shocks which piers receive, together with the shocks from traffic, and from merchandise falling while being handled on the pier. The slab described in the paper has been tried out and its thickness is slightly greater than that used in the original piers; any future change, if ever made, would be to increase it.

It may be interesting to many of the members of the Society to mention that another governing principle with the Dock Department engineers, in evolving a type of pier leading to the use of a reinforced concrete deck, was the elimination of any possible patented process. Municipal contracts in New York must be prepared so as to avoid the use of any patented processes, in order that competitive bids may be received to produce the structure under the design presented. This is not only a barrier, but is troublesome throughout the whole preparation of the contract and while the work is under construction. The Department, therefore, has evolved a type which, at least, has not caused any trouble from this source; it simply calls for a prescribed size of rod and spacing, or their equivalents in other shapes, on which a bid can be made by anybody manufacturing steel for reinforced concrete.

Mr. Taft's type, however, has many good features, and might well be used by private corporations desiring a permanent structure, the writer's only objection to it being the thin slab for heavy traffic. This is not such a serious factor as to detract from its usefulness, as the necessary increase in the thickness of this slab would not add greatly to the cost, although Mr. Taft makes quite a feature of this.

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FLOOD FLOWS.

Discussion*

By Robert E. Horton, M. Am. Soc. C. E.

Mr. Horton.

Robert E. Horton, M. Am. Soc. C. E. (by letter).—On page 1057,† Mr. Fuller states that:

"As far as the writer knows, the first suggestion of the use of the element of time in a formula for flood flows was made by Allen Hazen, M. Am. Soc. C. E., in a memorandum written by him in 1910."

It is not disputed that the idea of the use of the element of time in the discussion of flood flows may have occurred to Mr. Hazen independently, but in deference to the late George W. Rafter, M. Am. Soc. C. E., the writer feels obliged to deny that the idea originated with Mr. Hazen. This idea, as to its application to floods and also to other hydrological data, was suggested to the writer at least as early as 1896, or 14 years before it was suggested to Mr. Fuller by Mr. Hazen.

It was Mr. Rafter's desire that the writer should prepare the data for a paper on the application of the Theory of Probability to Flood Flows and other hydrological data, and the matter was discussed frequently and fully during the years 1896 to 1904. It appeared at that time, however, that stream flow data were so meager as to make it desirable to defer the completion of the studies. The writer continued the studies, as occasion arose, using the ordinary law of probability where the data were too meager to permit of plotting a satisfactory special probability curve on logarithmic paper.

* Continued from December, 1913, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., May, 1913.

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In 1896 the writer had prepared, at Mr. Rafter's request (by letter dated October 25th, 1896), an analysis of the rainfall and run-off of the Upper Hudson Drainage Basin, in which the probable average interval of recurrence of run-off less than the observed minimum was determined by the Theory of Probability. This and similar phenomena were analyzed by the ordinary method of probability, through the use of the so-called Gaussian Law of Error.*

It was recognized at that time that the Gaussian Law did not represent accurately the law of frequency of recurrence of many hydrological events, and an effort was made first to find a better general law† and to deduce individual laws. For the latter, the method of plotting flood records on logarithmic paper was tried, and proved so promising that, between 1900 and 1908, many logarithmic probability curves were plotted by the writer and corresponding formulas deduced, involving time as an element; and the methods and results have been applied extensively in professional practice by the writer for a period of several years.

He also applied the method of analysis of flood data by logarithmic special probability curves to the determination of maximum navigable stages, capacity of waste-weirs, etc., in connection with his work as Resident Engineer in Charge of the Bureau of Hydraulics of the New York State Barge Canal.

These results, after discussion with other engineers of the Department and with the Advisory Board of Consulting Engineers, were used as a basis in the design of work costing some millions of dollars. In this work the writer adopted 100 years in most cases as the average interval of recurrence of a flood of the magnitude to be made the basis of design. For the determination of relative frequencies of floods of different magnitudes on small torrential drainage basins in the Upper Mohawk River area, for which flood records were mostly wanting, the writer used as a basis of comparison in 1906 logarithmic flood discharge formulas which he had previously deduced for the Neshaminy, Tohickon and Perkiomen, as follows:

$$Q = 30\ T^{0.25} \text{ for the Neshaminy,}$$
$$Q = 30\ T^{0.27} \text{ " " Perkiomen,}$$
$$Q = 40\ T^{0.25} \text{ " " Tohickon,}$$

in which Q = flood discharge equalled or exceeded in an average interval of T years, in cubic feet per second per square mile.

These formulas were based on 20 years' records of the streams, covering the period, 1885-94. They give the results shown in Table 37.

* Report of State Engineer and Surveyor of New York, 1896, p. 841 *et seq.*

† In a letter dated August 11th, 1897, Mansfield Merriman, M. Am. Soc. C. E., suggested to the writer the use of a function of the form $y = (x + a)^2 (x - b)^2$ for this purpose.

Mr. Horton.

The writer also deduced the following general formula, applicable to these and similar drainage basins:

$$Q = 4\,021.5\frac{T^{\frac{1}{4}}}{A},\quad T = \sqrt[4]{\frac{Q\,A}{4\,021.5}}$$

or, as a logarithmic formula,

$$\log.\ Q = 3.6043881 + \frac{1}{4}\log.\ T - \log.\ A,$$

in which A = drainage area, in square miles; other notations as in Table 37.

TABLE 37.

Stream.	Drainage area, in square miles.	Interval, T, in Years.				
		1	10	25	50	100
		Cubic feet per second per square mile.				
Perkiomen	152	29.7	49.6	60.5	70.5	82
Neshaminy	139.3	29.4	55.0	71.0	85.5	104
Tohickon	102.2	40.5	71.0	89.0	105.0	125

In view of the foregoing considerations, the writer fails to discover wherein there is any proper basis for a claim of originality for this method, as far as the use of time as an element in flood formulas is concerned, as set forth in the paper. The writer does not claim any great breadth of applicability for the general formula just given, but believes that factors other than area modify the flood discharge of streams so profoundly that it is better, wherever possible, to derive individual formulas or flood-frequency diagrams for each stream. Following this idea, he deduced flood-frequency diagrams in 1906-08, for use in connection with the New York Barge Canal work, for the Mohawk, Hudson, Oswego, Seneca, and other streams. He has deduced and used similar special probability curves and logarithmic formulas to determine the law of frequency of recurrence of various other hydrological events, such as minimum seasonal rainfall, duration of periods without rain, relative probability of floods in different months of the year, minimum run-off of streams, storage volume required to maintain a constant supply from a stream, rain intensity of various durations, etc.

Attention may properly be called to another study, utilizing the same principles, which seems to antedate Mr. Hazen's suggestion of 1910, namely, the rain intensity diagram and formulas of J. de Bruyn-Kops, M. Am. Soc. C. E., for Savannah, Ga.*

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 248.

Mr. Horton.

With reference to the Neshaminy, Perkiomen and Tohickon flood formulas derived by the writer in 1904, as given in Table 37, it should be noted that these were derived in such a manner as to include all floods exceeding a given magnitude, and not merely the greatest single day's flood of each year. The writer's usual method of deriving a formula for the frequency of annual floods which will equal or exceed a given limit is similar to that described in Table 5, Part *c*, except that in his analysis Mr. Fuller discards all floods less than the mean, and works from ratios. The writer utilizes all annual floods, both greater than and less than the mean, although it is sometimes necessary to use a different flood formula for floods of low intensity. He also prefers to work directly from the original data rather than by reducing the data to the form of ratios.

Mr. Fuller says of his general formula, as given on page 1014:*

"$Q = CA^{0.8}$ $(1 + 0.8 \log. T)$, in which Q is the largest 24-hour average rate of flow to be expected in a period of T years."

Tracing out the derivation of this formula (given on pages 1023-1027*), it appears that Q is there defined as the "average maximum flood," and the meaning of this term appears to be quite different from that first given.

Assuming that Mr. Fuller's formula represents the average magnitude of floods having an average interval of recurrence greater than T, it may also be said that it represents the magnitude of a flood which will be equalled or exceeded at some average interval greater than T, but that this interval is unknown and cannot be determined from his formula. This illustrates the indefiniteness of his formulas when applied to the problem of flood determination, in the form in which, from the writer's experience, this problem usually occurs.

Again, in the case of Tohickon Creek, Mr. Fuller apparently does not mean by his formula for the average maximum flood of that stream, $Q = Q$ (Ave.) $(1 + 0.76 \log. T)$, that we may expect a flood at least as great as, or in other words, equal to or greater than 6 300 cu. ft. per sec. once in 5 years, for his own analysis of the same data (Table 5, Part *c*) shows that we may expect a flood to equal or exceed 6 300 cu. ft. per sec. on an average only once in about 10 years, or one-half as often.

On reading the paper carefully, the writer is of the opinion that the author has failed to set forth clearly the meaning of his own deductions. The writer's understanding of the matter is as follows, referring to Table 5, Part *a*, for Tohickon Creek.

The average magnitude of maximum floods having average intervals of recurrence between 5 and 25 years is 1.53 times the mean flood, or 6 300 cu. ft. per sec.

**Proceedings*, Am. Soc. C. E., May, 1918.

Mr. Horton.

The average magnitude of floods having intervals of recurrence between 2.50 and 25 years is 1.29 times the mean flood, or 5 310 cu. ft. per sec., and so on.

The results of the author's general formula bear the same interpretation, except that it is probably approximately true, from the method of analysis used, that Q, in the formula, represents approximately the average volume of all floods having an average interval of recurrence greater than T, it being here assumed that the formula is correct for the particular stream in question.

What Mr. Fuller has really deduced is not the average magnitude of the floods having, for example, 25 years or 12.5 years average interval of occurrence. Statistically speaking, there can be only one magnitude of the maximum flood having for example 25 or 12.5 years average interval of recurrence. The appellation "average maximum flood" of a given interval of recurrence seems to be misleading and inaccurate. If we had a long enough record, and were to divide it up arbitrarily into 25-year periods, and were to take the average of the maximum floods of each period, we would get a quantity having some semblance to a maximum average flood of 25 years interval, but the writer fails to see any practical value in such a result. Furthermore, the result would obviously be very likely to be affected by the mode of dividing up the record, which condemns the method.

Take, for example, the 5-year Tohickon flood of 6 290 cu. ft. per sec. The author does not mean that we may expect a flood of just this magnitude on an average of once in 5 years. As a matter of fact, the probable average interval of recurrence of a flood of just this magnitude is very much greater than once in 5 years.

The writer cannot see that results in the form of the "average maximum flood" are of much practical use, but, if any one wants them, they are derived more easily and directly by the method shown in Table 5, Part *c*, than by the method used by the author.

In addition, it would seem that the method of Table 5, Part *c*, when properly applied, has several other decided advantages.

1.—It is simpler of application, requiring three columns instead of five.

2.—It gives directly the average interval of recurrence of a flood equal to or greater than a given magnitude. This is a factor of great importance, and has direct application in designing spillways, determining high navigable stages of waterways, etc.

3.—It enables the average interval of recurrence of a flood lying between any two given magnitudes to be readily determined.

For example, from Table 5, Part *c*, the average interval of Tohickon floods at least 1.21 times the mean, or 4 968 cu. ft. per sec., is five years. Their probability is, therefore, $\frac{1}{5} = \frac{5}{25}$.

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The average interval of floods of at least 1.45 times the mean, or 5 958 cu. ft. per sec., is 8.33 years. Their probability is, therefore, $\frac{1}{8.33} = \frac{3}{25}$. The probability of a flood between 4 968 and 5 958 cu. ft. per sec. is, therefore, $\frac{5}{25} - \frac{3}{25} = \frac{2}{25} = 0.080$, or the probable average interval of recurrence of a flood between 4 968 and 5 958 cu. ft. per sec., or of a flood having an average of about 5 460 cu. ft. per sec., is 12.33 years. Now, if Table 5, Part *a*, and the formulas deduced by the method there given, mean what the author says they do, would we not expect floods of this average magnitude to occur at intervals of about 3.57 years, instead of 12.33 years, as actual experience shows to be the case? This illustrates what seems to be a fatal error in the author's method of analysis.

It would appear that the relation between flood magnitude and frequency, when expressed in terms of either "average maximum flood" or "median flood," is not only very indefinite and difficult of comprehension, but is apt to be very misleading, if it is not indeed practically meaningless. It does not directly convey the information which the engineer usually most desires, for example: If a spillway has a capacity of 1 000 cu. ft. per sec., how often on an average will its capacity be exceeded?

Further, in Tables 4 and 5 for Tohickon Creek, the author has arbitrarily excluded from consideration all floods which did not exceed the average yearly one of 4 117 cu. ft. per sec. He has excluded thirteen floods and used eleven. Instead of following the rule "of making the best possible use of all the available information", he has made no direct use of considerably more than half of it. What the author was seeking was, it may be assumed, to find the law defining the relation between the magnitude of floods and their relative frequency of recurrence. If one were seeking to determine the law of velocity of falling bodies, and had 25 observations of the time required for different weights, varying from 10 to 60 lb., to fall through given heights, it would hardly be considered good scientific method to discard *a priori* all observations on weights of less than 35 lb.

Mr. Fuller's method of working with ratios (Table 5), instead of actual flood volumes, seems to be confusing and cumbersome. The data which the engineer has to start with are actual flood volumes; what he desires to determine, as a rule, as in designing a spillway, is a flood volume which will not be equalled or exceeded on an average oftener than once in a given interval, say, 100 years.

If we had two drainage basins, *A* and *B*, which were similar figures, with similar drainage systems, or "stream nets" as the German hydrologist calls them, and identical in geology, soil, topography, slope,

culture, and climate, but if A were larger than B, and if Q_A and Q_B were the true maximum 24-hour floods to be expected on these streams in an average interval of T years, then the difference between Q_A and Q_B would be the difference in flood-yielding capacity due to area alone. Mr. Horton.

In practice, however, it often happens that one area much larger than another does not yield a materially larger flood. If the data of such heterogeneous basins are plotted together in terms of area as abscissas, it appears to the writer that the results will be so discordant as to render the conclusions drawn therefrom of doubtful value.

Here we come squarely against the question of the physical, and not merely the statistical, aspects of the question. The latter only are covered by Mr. Fuller's discussion.

In the climate of the North Atlantic States local thunder storms nearly always produce maximum floods on very small areas, though only broad general cyclonic storms produce floods on the larger and medium large rivers, like the Hudson, the Susquehanna, and the Delaware. Now, thunder storms produce rains of great intensity but short duration. Broad cyclonic storms produce rains of low intensity but long duration, so that the relative flood intensity for different areas is not wholly determined by the relative times required for the water to reach the stream, or by the physiographic or cultural conditions, but partly by purely meteorological conditions.

The paper contains a valuable collection of flood data. It would have been easy to include the dates of the maximum floods, and these would be of use to engineers in comparing simultaneous flood intensities at different places. The writer cannot help feeling a sense of regret that so much labor has been expended in analysis of flood data along lines that do not seem to be either the most scientific or the most useful to engineers.

The ambition to place the results of one's investigations in the hands of the Profession at the earliest date is laudable, if indulged in with moderation. The writer's only excuse for withholding from publication the results of his own studies along similar lines for so long a time is that he thought it better to try the method out in practice first, rather than run the chance of punishing the Profession by the publication of that which might later prove to be premature.

The writer happens to have collected all the data contained in Table 13. Many of the floods there listed have subsequently been exceeded.*

* Reports on Hydrography (by the writer), State Engineer's Reports, 1900, 1912; also paper on "Effects of Recent Flood on New York Streams," *Engineering Record*, April 12th, 1913.

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CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN. Discussion.*

By Messrs. C. E. Gregory, Henry H. Quimby, A. C. Janni, Philip Aylett, and L. J. Mensch.

Mr Gregory.

C. E. Gregory, Assoc. M. Am. Soc. C. E.—The speaker was the Designing Engineer in charge of the design of bridges built by the Board of Water Supply, as referred to in this paper, and wishes to make a few statements to amplify the relation of the work as done by the Board of Water Supply to this paper, and to put others who were concerned in the design of these bridges in a right position in regard to it. The first point he desires to make is this:

The authors reach the conclusion that in practically all cases the three-hinged arch is more economical for long spans than the ordinary fixed-end arch. When designing the Traver Hollow Bridge, the speaker did not, and does not now, concur in such a conclusion. The three-hinged type was selected largely because of expected slight settlement in the foundations, rather than the economy of the type of bridge. The preliminary estimates indicated that a three-hinged arch would be more expensive than a fixed-end arch.

The economy of the divided pier is brought out by the authors, but the manner in which the design of the piers of the Rye Outlet Bridge was arrived at differed considerably from that described in the paper, although meeting the assumptions of the authors. The interesting and unusual feature of this bridge is that it is an arcade of five reinforced concrete arches supported on exceptionally high piers. It was found that the yielding of the top of a high pier of ordinary shape under the unbalanced live load arch thrust (the height of the highest pier in this bridge being a little more than 100 ft.) would be sufficient to

*Continued from December, 1918, *Proceedings*.

Mr. Gregory. create considerable stress in the arch without exceeding the usual limits of stress or creating tension in the pier. The design was made to limit the yielding of the piers to that which could be safely and economically provided for in the arches. The entire arcade was treated as a complete elastic structure, and determination was made of what proportion of the unbalanced thrust from a loaded arch would be carried by the adjacent pier and by other arches and piers of the arcade. The shape and size of pier adopted was found to be necessary from calculations made as just outlined.

This work was done under the speaker's supervision by Mr. George L. Bennett and Orrin L. Brodie, M. Am. Soc. C. E., who were at the time assigned to Mr. Smith's staff, but worked on this problem under the speaker's immediate direction.

Mr. Quimby. HENRY H. QUIMBY, M. AM. SOC. C. E.—Experience in the design of concrete arches confirms the opinion expressed by Mr. Gregory regarding the relative economy of fixed and hinged rings, and the justification for the use of hinges. Computations for comparison of designs in several cases have failed to show any saving in cost of a three-hinged type over a fixed type when the ratio of rise to span was fairly good. Hinges are justified only when the foundations are unsatisfactory, or in cases where the arch is very flat. In a flat arch the crown movement due to change in length of ring from temperature is greater than in a high-pitched one, and the temperature stress may thus become great enough to warrant the use of a hinge to avoid it. The arches shown in the paper, however, are not flat, but have a good rise.

The authors give estimated quantities of concrete required for the arch rings in the two designs shown, and represent the fixed type as requiring 90% more concrete than the hinged. This is hard to understand. A fixed ring should require only a little more than a hinged ring. The depth at the crown must be about the same for each because the horizontal thrust is about the same, and any theoretical eccentricity, due to unsymmetrical loading, in the fixed ring will probably be fully matched by the effect of friction of the hinge in the other case, for the inevitable deflection caused by the weight of the floor imposed after the closure of the arch will be likely to produce the effect of eccentric thrust. At the quarter points the hinged ring will need considerably more depth than the fixed ring, and it will be only at the springing line that the fixed ring will be deeper and require more concrete. As the cost of effective hinges is quite considerable, the net result is generally the reverse of that reported in the paper.

If it be said that the greater depth of the fixed ring design in the paper was made because of temperature stress, the reply is that temperature stress necessarily increases with the depth of the ring, and adding to the depth because of such stress is somewhat like piling a

load on a horse's back to enable him to pull a heavier wagon—the poor legs may be overtaxed. Mr. Quimby.

It would seem, therefore, that there must be something wrong either with the estimates or with one of the designs.

The stone hinges shown in the paper give the impression of being likely to develop friction if they work at all, and this will entail eccentricity of stress in the arch ring, to that extent detracting from the only advantage of the hinge.

The matter of appearance of an arch is important in many places, and the bulging at the quarter that is characteristic of the ordinary hinged ring is distinctly ungraceful and displeasing to the miscellaneous eye which cannot understand the reason for it, while the eye of the designer who knows that the point of greatest shear is the point of greatest single stress, desires to see the arch thickest at the spring where that stress is the greatest.

One very important consideration in a discussion of stress in a masonry arch is the method of construction, and it is generally far more vital than temperature fluctuations. The old-fashioned method of laying up an arch ring from the springing line to the crown, with the attendant progressive deformation of the centering and consequent cracking of the earlier portion of the ring practically reduces the working thickness of the ring to a fraction of that designed, and may be the real cause of some failures that have occurred. The proper way to avoid cracks or initial stresses in the arch ring due to settlement or compression of the centering during construction is to build the ring in voussoir blocks with narrow key spaces between them. Then when all the blocks are made, and the centering has, therefore, nearly its full load, after the blocks have seasoned and shrunk, cast the keys. Of course, if the arch is small enough to permit the casting of the full length of the ring in one day—say within 10 hours—the separation into blocks will not be needed, because the first portion cast will be still plastic enough when the ring is closed to adjust itself to the deformed centering. It is impracticable to eliminate initial stress wholly, for any of the dead load that is added subsequently to keying will cause some deformation of the ring and consequently initial stress somewhere; but this should never be enough to be serious.

The claim advanced in the paper that concrete will weather better than good sandstone, and almost if not quite as well as granite, is hardly to be expected from an experienced observer of concrete work. If the authors know of any formula that will produce concrete with such a desirable quality, they should give it to the Profession. It is a poor quality of building stone that will not hold its surface and its edges better than the best concrete. The statement in the paper sounds a little over-enthusiastic. The friends of concrete will promote its use more effectively by avoiding extravagant claims for it.

Mr. Quimby. Another point in the paper that seems not entirely clear is the claim of marked advantage in the I-shape of arch rib. The arch ring is a column—not a beam. If the curve be true and the line of thrust be central, there is absolutely no beam action. Of course, the width—horizontal thickness—of the rib, and its depth or vertical thickness, should both be theoretically adjusted to make a column that will fail by crushing rather than by buckling, and this will require some minimum width. As regards lateral buckling, any given width is as effective at the mid-depth as at the flanges, and, in any case, suitable transverse bracing should be provided. The method of estimating the saving due to the coring out of the concrete at the sides is hardly fair, because a very large part of the cost of such concrete is in the support of it, and any increase or decrease in quantity will be proper to be charged at only the cost of the material and placing. Besides, the extra cost of the additional form work necessary for the paneling will materially reduce the seeming economy in material.

Replying to the question of a member as to the observed actual range of temperature in concrete arches: Two bridges, constructed within the past 5 years, with electrical resistance thermometers embedded at the mid-line, show temperatures in the concrete ranging far less than is usually assumed. The depth of embedment in one case is 2 ft. 6 in. and in the other, 4 ft. 9 in. The former is a spandrel filled arch and the other is an open spandrel. The maximum range of temperature recorded thus far has been from 31° Fahr. in winter to 73° in summer, and measurements of change in elevation of crown made at the same times show a rise and fall computed to correspond to a change of temperature throughout the arch ring of 45° Fahr. between the extremes of cold and heat.

Mr. Janni. A. C. JANNI, M. AM. SOC. C. E.—The statement, among others, made concerning the quickness, simplicity, and certainty of the design for three-hinged arches, deserves careful investigation, and should not be accepted off-hand, or as a matter of course.

Therefore, it is not without interest to recall what both theory and experience teach with reference to the equilibrium of this particular kind of construction, and to ascertain to what extent a designer may trust his results.

If S and S_1 (Fig. 5) are the two contact surfaces of the left hinge of an arch, P being the tangent point, and PN the normal to the tangent, T, it is known that as long as the line of action of a force, F, acting on the hinge, is contained within the angle APD (this angle being twice the angle of friction, ϕ, for the material of which the hinge is made), the system will be in equilibrium, provided, of course, the force, F, is a compressive one with respect to the tangent, PT. Similar remarks may be made regarding the surfaces, S' and S_1', of the hinge at the key.

Mr. Janni.

Therefore, as long as the two components of a force, P, acting on the system, will keep within the two angles, APD and $BP'D$, the system itself will be in equilibrium, and the points, A and B, are to be regarded as the extreme limits of this equilibrium position.

In other words, the system will be always in equilibrium under the action of any force the line of action of which cuts the area $ACBD$.

The foregoing demonstration is very plain, but its foundation is rather indefinite; in fact, it is all based on the assumption that the angle, ϕ, is a known quantity, and, even if that is true, there are two solutions, according to the theory, that satisfy the conditions of equilibrium and, especially for an arch of large and flat span, give quite different results.

Concerning the friction laws, it can be said that, outside of those values of ϕ reported in all engineers' handbooks, which for every-day purposes are tolerably reliable, those laws are not well known at present.

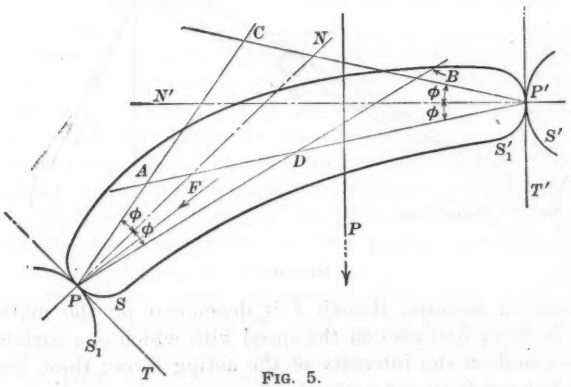

FIG. 5.

The history of the researches on friction is very brief. Amontos, in 1699, started for the first time a research on the friction of some machine of his, without, however, throwing any important light on the general matter.

In 1781, Coulomb took up the researches on this important problem and announced to the world his results, giving out laws on friction, as well as values of ϕ for different materials.

In 1785, Vince started his researches on the same question and rejected, as unreliable, Coulomb's laws and values of ϕ.

In 1829, G. Rennie, who made tests under about the same conditions as Coulomb, found more or less similar results.

Finally, in 1831, Morin presented before the "Académie des Sciences", in Paris, the results on his friction tests, and, though he

Mr. Janni.

confirmed Coulomb's fundamental laws, he found numerical values for ϕ which were very different. It is worthy of note that, though Morin made his tests on a larger scale than Coulomb, he did not go farther than a pressure of about 5.2 lb. per sq. in.

As far as the speaker knows, these are the only works on the subject which stand high on account of the illustrious names of their authors; unfortunately, they do not agree among themselves.

Modern investigations, however, without adding anything definite, have found that the friction law, as given by Morin,

$$R = fN$$

where: R = Resistance due to friction;
N = Normal component of the force acting on the body; and
f = A numerical coefficient depending on the materials in contact;

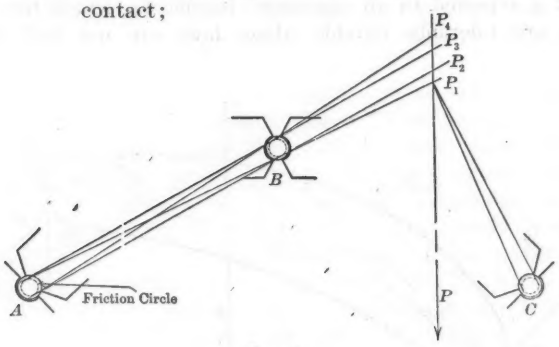

FIG. 6.

is not complete; because, though f is dependent on the materials in contact, it is dependent also on the speed with which one surface moves on the other and on the intensity of the acting force; these laws, however, have not yet been determined.

The only thing known at present is that the value of ϕ, at the start, is between 1.5 and 3 times its value during the movement, and this is quite a wide range when one considers the certainty of results.

Usually, the hinges are metallic, or, even in stone, they have a shape which might be regarded as a hinge; the problem then is solved, as is well known, in the following manner:

Let A (Fig. 6) be the left hinge of an arch; drawing the friction circle with the radius $r \sin. \phi$ (where r is the radius of the hinge and ϕ the angle of friction), it is known that all forces having lines of action which cut that circle will not move the hinge about its center, and all forces having lines of action tangent to that circle will be on the limit position of equilibrium, provided each of these forces has a tendency to compress the hinge against its groove.

Mr. Janni.

If B is the hinge at the crown, it is known that the four common tangents to the two friction circles are limit positions of equilibrium, therefore, according to graphic statics, the problem already admits of four solutions.

Taking now into consideration the third hinge, C, if P is a force acting on the system, and P_1, P_2, P_3, and P_4, are the points where the line of action of P cuts the four tangents, from each of those points can be drawn two tangents to the friction at C, which, according to the theory, will resolve the problem.

For the sake of brevity the hinge at B has been regarded as of the same kind as those at A and C.

Apart from the several solutions given by the theory on the equilibrium of this kind of construction, here, too, the indeterminations, resulting from friction, must be considered.

The friction circles can be drawn quite accurately for machinery, or when it is a question of pieces well prepared for laboratory tests, for which test after test has been made so that the whole question is somewhat settled.

Concerning the friction in bridge hinges actually in operation, however, on which the pressure is often so great as to wear off the material at the contract surfaces, it is to be said with regret that there has not been a single test known to the speaker.

It can be seen, therefore, that the quantity, $r \sin. \phi$, has not, by any means, a known and determined value; in fact, it is rather arbitrary. In other words, the set of four points, P_1, P_2, P_3, and P_4, can be substituted for another set of four points, provided they satisfy the requirements of graphic statics.

Although all the stated indeterminations (and there are others) do not prevent a designer from patching up some kind of design for a three-hinged arch, it is not possible to accept the statement concerning the reliability of such design.

The hinges, however, in a concrete or masonry arch can be used with advantage sometimes during the construction of the arches, when there is no steel reinforcement to take up the tension at the springing line and at the key, due to a possible excessive settlement of the centering; then, after the centering has been removed, the hinges should be concreted. Thus the arch behaves as a three-hinged arch, as far as the dead load is concerned, and as a rigid arch for the live load and temperature. In that instance the hinges, free from the enormous pressure of the dead load, can easily accompany any movement of it, being concreted afterward when their behavior begins to be extremely doubtful.

The granite railroad bridge at Morbegno, Italy, with 229.65 ft. span and 32.8 ft. rise, has been built in accordance with the foregoing method.

Mr. Janni.

In comparing concrete and stone bridges, the authors rightly state that one of the advantages of the former is that it is stronger and much more reliable, because of the absence of joints; therefore it is not clear why the paper is a special plea for the three-hinged arch, which has three joints, as compared with the rigid arch, which has none.

Concerning the statement that the trial design for a fixed-end arch takes several days for each analysis, it can be said that it takes no longer than the trial design for a three-hinged arch, and possibly less.

The stresses in the arch with fixed ends, due to dead load and temperature, can be calculated in a day's work easily.

A new method of designing an arch, based on the rather recent theory of the "Ellipse of Elasticity", is described in a paper by the speaker.* This is a purely graphical method and holds, irrespective of the geometrical form of the axis of the arch, irrespective also of the law of variation of the cross-section of the latter, and of the assumptions of loading. In other words, it is possible, by that method, to draw the lines of influence for moments, for any section of the arch, before the loads are applied on the arch, and, as a consequence, the most prejudicial hypothesis of loading for the chosen section is given at once, without any trial.

In that paper, in addition to the theoretical part, the speaker showed a practical application to an arch which he had designed and built by that method.

In contrast with the unreliability of the results in a design for a three-hinged arch, it is not without interest to relate that for a fixed-end reinforced concrete arch, built in France, having a span of 318 ft., the difference between the stresses calculated and the actual stresses reached a maximum of 14.7 per cent.

Every engineer fully appreciates the importance of these results, especially when he considers that, in steel bridges of the best design, that difference reaches 25%, and, when designed on the assumption that there is no friction at the joints, the difference may be more than 100 per cent.

The horizontal thrust, due to temperature changes, is directly proportional to the coefficient of expansion, to the change of temperature in degrees, to the modulus of elasticity of the material, and to the span (measured at the spring points of the geometrical axis of the arch); but it is inversely proportional to the inertia moment of the whole elastic arch with respect to the horizontal (if the arch has its spring points at the same level) passing through a certain point called the center of gravity of the elastic arch.

If, for the same span, arches with different rises are drawn, it can be seen that, although the factors directly proportional to the hori-

* *Journal*, Western Society of Engineers, May, 1913.

zontal thrust remain unchanged (the variation of the span of the geometrical axis may be disregarded), the moment of inertia of the arch increases rapidly with the increase of the rise of the arch; so that the effects of a change in temperature diminish in importance with the increase of the rise. Mr. Janni.

The shrinkage of the concrete due to setting and that due to the dead load are two important causes of stress to be taken care of, especially the former. If the arch is very flat and has a large span, the best and most economical way to take care of the shrinkage due to setting is by neutralizing its effects, as much as possible. A span of 170 ft. with 17 ft. rise, and two spans of 140 ft. with 14 ft. rise, designed by the speaker, have been built, breaking the arches into voussoirs each about 20 ft. long, and leaving between each two consecutive voussoirs a small voussoir, or key, of 2 ft. The voussoirs were cast first and the keys were not cast until the last concrete of the voussoirs was 10 days old, thus avoiding, for the 170-ft. span, for instance, important tensions on the intrados at the key and on the extrados at the springing line, especially during the winter, due to a shortening of $\frac{7}{8}$-in. of the geometrical axis of the arch, which otherwise would have taken place.

If, as the authors state, an average pressure of 12 000 lb. per sq. in. on the crown hinge of that arch was calculated, it must be inferred that the maximum pressure on the hinge was about 24 000 lb. per sq. in. at the theoretical point of contact with its groove, and it may be assumed that it was reduced to zero at each end of the groove.

The question then arises: At such a vital point of a bridge may the material be submitted to work at a rate which is never allowed for such material at other much less important points; and that without discussing the practical inadvisability of trusting on a $2\frac{1}{2}$-in. space the stability of such a comparatively enormous mass of material?

In dealing with such pressures, however, there is no longer any question of the possibility of assuming that at that point there is a hinge, ready to work; it is no more a matter of design, but of guesswork.

When a hinge of such small diameter is placed at the key, a malicious attempt to wreck the span could be successfully carried out.

Another kind of hinge—fortunately, very seldom used—is a ball-and-socket joint of cast steel consisting of a hemispherical protuberance fitting into a hemispherical cavity. Any horizontal displacement of the crown of the arch, for a structure of this kind, cannot be contemplated seriously; there is no reason for using such a hinge. In the construction of a three-hinged arch, the use of a ball-and-socket hinge means simply the addition of more friction at that joint.

Having an arch with only two ribs, and that arch hinged with the ball-and-socket type, then, at each joint cross-section there will be

Mr. Janni.

at least two hinges. Admitting, for the moment, that a horizontal displacement of the crown might take place, it will be admitted also, that it will be very small.

On account of the rigidity of connection between the two ribs, however, there is no possibility of a horizontal rotation of both hinges, even disregarding friction and admitting, of course, the elastic deformation of the whole system; but the effect of that displacement will be only to decrease the pressure on one hinge at the expense of the other.

However, as the use of a hinge of that kind shows that the designer assumed the possibility of a really tangible horizontal displacement of the key and the consequent horizontal rotation of the hinge, it is not without interest to ascertain what would happen to the arch if a horizontal rotation took place at the key, no matter how small it might be, provided, however, that it be a tangible quantity.

Assume that the hinge has a horizontal rotation of only 10′, and that the two hinges are 30 ft. apart; then, while one hinge presses in its socket during a rotation of 10′ (always disregarding friction), the other would be lifted out of its socket a distance of about 1 in. Although this will not happen, on account of the elastic deformation of the system, it is fair to admit that, at that joint, the horizontal thrust is zero (without speaking of the pressure concentrated on the rotating hinge).

What happens at the key, according to the designer's assumption, will happen also at the springing line, but the order will be reversed. Then the question arises: Did the designer calculate the stresses in his elastic system under such abnormal conditions of equilibrium? If he did, it is certain that he will never advocate the ease, speed, and certainty of a design for a three-hinged arch, especially with ball-and-socket joints; if he did not, then his enthusiasm for structures of this design is quite natural.

M. Mesnager, wisely concerned about the secondary stresses arising from the use of the hinges in arch bridges, and for which there is no way, at present, to determine their exact amount, thought that, a half-rigid joint "semi-articulation", the working behavior of which could be easily determined, would be preferable to the usual hinges.

For that purpose, several years ago, under his personal supervision, the Testing Laboratory of the "École des Ponts et Chaussées" started researches on a joint, Fig. 7, built in reinforced concrete according to his plans. Without going into details concerning his tests, it is enough to repeat one of his remarks:

"Le moment qui a été nécessaire pour fair fléchir la semi-articulation quand elle était noyée dans le béton, n'était pas négligeable."

Then he concluded by saying that it was possible, perhaps, to free the joint, *J*, from concrete and fill the remaining space with some kind of non-acid asphalt.

Evidently, a hinge of this kind, if developed properly, could not be expensive, and, at the same time, would eliminate the secondary stresses depending on friction, at least. Mr. Janni.

Concerning the saving of material by building the ribs of I-shaped section, it may be remarked that such a saving, in a general way, does not mean, necessarily, a saving of money. It would be rather absurd to think that a marble statue costs less than a block of the same material having the same external dimensions, only because the actual volume in the statue is less than that in the block. In addition, however, to all doubts expressed by other speakers concerning the cheapness of an I-shaped rib, as compared with the rectangular rib, another remark may be made: The three-hinged arch, on account of its lack of rigidity, requires a stronger system of stiffeners than that needed by a fixed-end arch, and the quantity of concrete in the stiffeners for the former must, necessarily, be greater than in those for the latter; therefore the statement advanced by the authors, that the quantity of concrete was the same in each case, cannot be accepted.

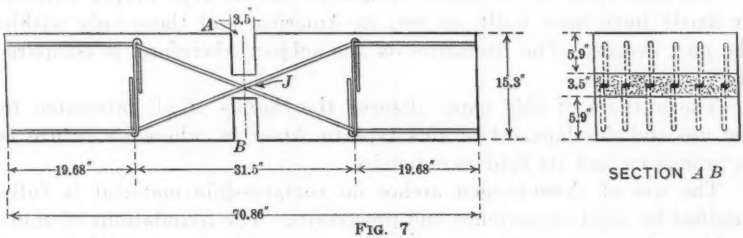

FIG. 7

In conclusion, it is to be noted that the statement concerning the complication of design for a fixed-end arch and the ease with which such an arch can be upset by a slight movement of one of its supports, is not altogether justified. This complication of design, so much complained of, is justified only if the designer expects to calculate the arch as simply as he can pick out, from the "Carnegie Handbook," the I-beams for a floor.

Concerning the latter remark (which was correct several years ago) it can be said that, by the theory of "virtual work" or, more elegantly and speedily, by the method of the "ellipse of elasticity", the designer can easily take care of such dreaded displacement of support, and even of a yielding of the soil on which the footings rest.

There is no doubt that the design for an arch is one of the most delicate problems of engineering; but, after all, it may be done without any great trouble if the designer is sufficiently familiar with his work.

The theories used are not monopolized by a few persons, but are quite accessible to any one of training and application. Fortunately for the designer, those theories cannot be reduced to "rule-of-thumb",

Mr. Janni. for several reasons. If an engineer, satisfied with what he was taught at school, does not keep in touch with the daily progress of science, he runs the risk, nowadays, of being behind the times. If, for instance, in designing an arch, he still assumes first that the arch is entirely loaded and then, that it is half loaded and half unloaded—still believing that, in this way, he ascertains the maximum stresses in that arch—he is flatly wrong; those assumptions were made several years ago, because there was then no means of ascertaining, conveniently, the most prejudicial hypothesis of loading; at present, however, that method of computing stresses is antiquated as well as inadequate.

Mr. Aylett. PHILIP AYLETT, ASSOC. M. AM. SOC. C. E. (by letter).—In America, the ribbed arch, especially the three-hinged type, has not received that recognition of its merits which it deserves. Its economy and advantages are better known and more highly appreciated in Europe, where it has been used extensively under almost all conditions of site and traffic, and in great variety of form and design.

Comparatively few concrete bridges of the rib type proper (hinged or fixed) have been built, as yet, in America, and these only within the past decade. The literature on the subject, therefore, is comparatively scant.

The authors of this paper deserve the thanks of all interested in the use and development of this type in America, where its future is so promising and its field so extensive.

The use of three-hinged arches on compressible material is fully justified by ample experience and precedents. The foundations of abutments and piers in such cases, however, demand more careful investigation, study, and design. A fixed or a two-hinged arch may also often be built with entire safety on compressible material, provided the foundations are properly designed and built, including the use of batter piles and other methods of taking up thrust. This also is justified by numerous structures under such conditions now extant, both in Europe and America.

The seeming economy of the rib of I-section, as compared with the rectangular, will not show so vividly in practice as on paper. This saving in concrete in the I-section will be largely, if not entirely, offset by the greater expense caused by the additional material and labor involved in making and placing the I-forms and depositing the concrete therein. Forms for such sections also necessitate the cutting up of the lumber into many short lengths, thus making much of the material unfit for other use. For instance, Fig. 8 shows an I-section form designed for the purpose of economizing in material and in the labor of making, placing, and dismantling it. This form is also capable of repeated use, as it is built in sections with this in view. Yet the

additional material and labor involved, in comparison with forms of rectangular section, is apparent at first glance. Mr. Aylett.

In addition, the I-section rib invites and makes possible injury and defects to the permanent structure from which the rectangular section is exempt. No one can foresee or forestall settlements in falsework or centering from timber shrinkage, "taking up", "biting", col-

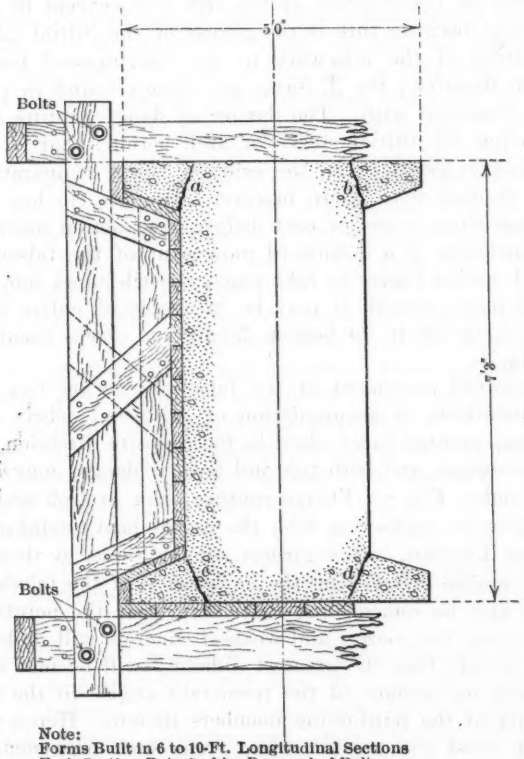

FIG. 8.

umnar shortening, foundation movements, expansion, contraction, etc., etc. Falsework or centering moves upward as well as downward on account of the unsymmetrical placing of the load, or in cases where the falsework becomes wet by rain or flood after being thoroughly dried out. A rectangular arch rib is capable of sustaining more settlement or upward movement of the falsework without injury than is possible with an I-section, because of the projecting flanges of the

Mr. Aylett.

latter. Settlements or movements of the falsework prior to the removal of the I-forms seriously endanger these concrete flanges, and will probably cause rupture at or near their junction with the body or web of the section, as at *a, b, c,* and *d,* of Fig. 8.

From a number of cases, it has been observed that the maximum movements of falsework (both up and down) occur during or immediately after the construction of the arch ribs (except in cases of accidents, etc.); because this is the period of the initial adaptation or accommodation of the falsework to its superimposed load. During this period, therefore, the I-forms are always found in position and cannot be dispensed with. The danger of flange rupture is most imminent during the initial stages of arch action; that is, when arch action begins to assert itself, the concrete being comparatively green.

Flange rupture may occur, however, after the rib has passed into the self-supporting stage, or even before arch action asserts itself at all. For instance, if a downward movement of the falsework occurs just as arch action begins to take place, the rib offers some resistance to the movement, though it may be incapable of entire self-support. The result is rupture in the bottom flanges and cracks opening upward, as at *d,* Fig. 8.

If an upward movement of the falsework occurs (on account of swelling, distortion, or accumulation of drift and débris against the forms), flange rupture takes place in the opposite direction, the cracks opening downward, and both top and bottom flanges may be affected, as at *a, b,* and *c,* Fig. 8. Flange rupture prior to arch action is most likely to occur in connection with the use of heavy reinforcing members, such as I-beams, lattice girders, etc., as these, by their own arch action, are capable of resisting the movements of the falsework.

It must also be remembered that at or near the points, *c* and *d,* in the I-section, the poorest and weakest concrete will undoubtedly be found. It is here that the greatest difficulty will be met in securing good concrete, on account of the re-entrant angles in the forms, and the proximity of the reinforcing members thereto. Hence the flanges which must resist the greater stress from such movements are less capable of doing so, on account of the quantity and quality of the material.

The likelihood of flange rupture (especially after arch action in the rib) may be decreased, more or less, by avoiding a rigid connection between the sides of the I-forms and the supporting timbers below. Thus, in Fig. 8, such a precaution may be provided by simply omitting (on both sides) the lower bolts shown in the verticals, and using stop-blocks instead. Slip-joints thus formed would permit the downward movement of the falsework without injury to the flanges, provided the arch had attained the stage of self-support.

Mr. Aylett.

Although the authors affirm that "the shape of the piers" is a "most important point in the design", they appear to overlook this and other important points relating to the design and construction of bridge piers, especially in flowing streams.

In the comparative estimates of the braced and solid types of piers, apparently, only the difference in concrete has been considered. The greater length of coffer-dam required by a pier of the braced type (with or without diaphragm or foundation offset) should be considered. In constructing coffer-dams in flowing streams, the important thing is usually time, for more risk is attached to coffer-dam building in flowing streams than in any other operation connected with ordinary bridge building, especially where there is "ice or logs of wood moving rapidly".

The many-sided dam of the braced type pier requires much maneuvering of the pile-driver in order to effect the frequent changes of direction. This involves change of anchorage, backing, going forward, etc., etc., all by men who are not sailors, and while breasting a current, and with risk of disaster by flood.

The solid rectangular pier requires less coffer-dam, less maneuvering of the pile-driver, less exposure to flood at the most critical time, and, consequently, much less time and expense. These are among the important points which should be considered in pier design and construction, and cannot be overlooked in any estimates.

There are, however, other features of the authors' braced pier which appear so serious as to preclude entirely this type from consideration in streams of the character mentioned.

The authors state:

"This type [braced] of pier is especially economical for a bridge in running water in which there is likely to be heavy blocks of ice or logs of wood moving rapidly."

Even prior to 1913 streams of this character throughout the country indicated, by ample evidence and signs, the imprudence of obstructing their flow. Railway officials and engineers, having interest in many streams over extended territory, were among the first to be impressed. Frequent and alarming flood records in numerous streams were observed. Bridge piers which had given many years of service, without trouble or expense, became the causes of claims and litigation on account of damage to adjoining lands by soil stripping and deposits of mud, silt, and débris, resulting from stream obstruction and current deflection.

It was observed that, usually, mud, silt, etc., were deposited in greatest abundance where the water first left the banks of running streams. This occurred in most cases immediately below bridge piers or other current deflectors.

Mr. Aylett.

Railway men and others who have borne the responsibility (night and day) of traffic and maintenance of way during flood periods, know the expense and risk attached to ordinary solid piers in running streams carrying ice and drift. The braced pier recommended by the authors for such locations would prove a veritable drift-trap and, viewing the matter broadly, appears to be a most obstructive and expensive type. Cut-waters, though valuable safeguards, especially against ice, do not prevent the lodgment of all logs and drift, even when solid piers of comparatively less widths are used.

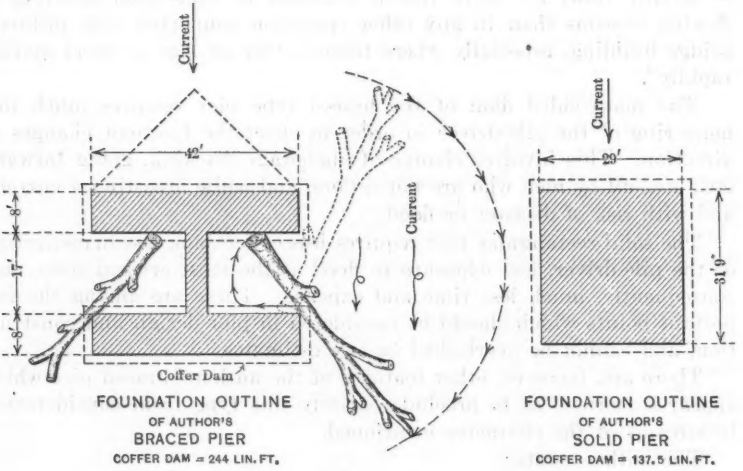

FIG. 9.

The authors' braced type of pier is exposed, not only to the accumulation of drift, etc., on its nose or up-stream end, as in solid piers, but will also catch and hold rigidly logs and drift in the space between the legs on both sides (Fig. 9). Drift thus trapped and held would not float and rise with the flood as it does with solid piers, and as it would be quickly submerged when caught, it would be almost impossible for an ordinary section gang, even with machinery, to dislodge it.

The buoyancy and enormous leverage of a mass of logs and drift, thus trapped, would exert a simultaneous lifting and wrenching action on the pier, increasing with the pressure of the rising flood on the growing mass held firmly before it.

Drift which collects at the noses of solid piers can always be removed by ice-hooks or pike-poles and floated away. Masses of drift cause deep scour around piers, and this would probably extend below the diaphragm, if its depth were only "down to where they [the pier-legs] enter the ground." If this should occur, heavy drift from both

sides might find its way under the diaphragm as well as the braces. It is difficult to reconcile the obstructive braced pier recommended by the authors with the nation-wide conservation movement of to-day, especially in the face of the experience of the West and Middle West in 1913, with its lessons so clearly, impressively, and profoundly taught. Mr. Aylett.

Amid the losses of lives and property, the bridge engineer's lesson stood out above all others—unmistakable and specific:

It was: Conservancy of Rivers by

(*a*) Avoiding unduly wide and obstructive piers;

(*b*) Reduction in number of piers by increase of span lengths;

(*c*) Increasing floor-level elevations.

These would facilitate the discharge and consequently help to lower the flood level. By so doing, other lives and property may be saved, and the sacrifices in the West and Middle West will not have been in vain.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—The statement of the authors that three-hinged arches contain only about half as much masonry as properly designed fixed arches is only true for bridges of very small rise, and the writer would be interested to see the calculations for the bridges shown in Fig. 3, as his own tables do not show such a difference. The fact is that the thrust in both types is practically the same. The bending moment caused by a concentrated load in a three-hinged arch is greatest when the load is 0.211 l from the abutment, and its value is about $P \times \frac{l}{10}$; in a fixed arch a concentrated load produces the greatest bending moment when placed 0.3 l from the abutment, and its value is about $P \times \frac{l}{16}$. Mr. Mensch.

It is customary to calculate a bridge for a uniform load, w, placed at its half span. The greatest bending produced in a three-hinged arch occurs at the quarter span, and amounts to $\frac{w \times l^2}{64}$; in a fixed arch it occurs at the point 0.3 l from the abutment, and amounts to $\frac{w\,l^2}{120}$; at the same time the moment at the abutment is $\frac{w\,l^2}{64}$.

There is nothing to justify the statement that there is a considerable saving in materials in a three-hinged arch over a fixed arch, unless the temperature stresses and the stresses due to the shortening of the arch and the shrinkage of the concrete play an important rôle, which is only the case in arches of comparatively small rise and large depth. Professor Howe, in his treatise on arches, showed that the thrust from a change of temperature in a parabolic arch of constant $\frac{d\,s}{I}$ may be found by the formula, $\frac{45}{4}\,\frac{E\,I\,c\,t}{f^2}$, when E = the modulus

Mr. Mensch.

of elasticity, I = the moment of inertia of the crown section, c = coefficient of expansion, t = change of temperature, in degrees Fahrenheit, and f = rise of arch; and that its point of application should be assumed at $\frac{1}{3}\,f$ below the crown. The writer finds that in a parabolic arch, as usually designed, the thickness of the arch increasing at the abutment to about $1\frac{1}{2}$ to 2 times the crown thickness, the thrust may be found by the formula, $24\,\frac{E\,I\,c\,t}{f^2}$, and its point of application may be assumed to be $\frac{7}{30}\,f$ below the crown. Let $E = 2\,000\,000$, $t = 50°$, $c = 0.0000055$, or $E\,c\,t = 550$.

From a change of temperature, the bending moment is found to be,

at the crown, $$\frac{24\,I}{f^2} \times 550 \times \frac{7}{30}\,f = 3\,080\,\frac{I}{f};$$

at the abutment, $$\frac{24\,I}{f^2} \times 550 \times \frac{23}{30}\,f = 10\,120\,\frac{I}{f}.$$

For a width of 1 ft. of the arch ring, $I = \frac{12\,d^3}{12}$, when d = the thickness at the crown; the moment at the crown $= 3\,080\,\frac{d^3}{f}$, and the moment at the abutment $= 10\,120\,\frac{d^3}{f}$.

If we consider that the moment of resistance of the crown section for a width of 12 in. (neglecting the reinforcement) $= \frac{12 \times d^2}{6} = 2\,d^2$, and that the moment of resistance at the abutment, when the thickness is twice the crown section $= \frac{12 \times 4d^2}{6} = 8d^2$, we can obtain for the stresses in the extreme fibers caused by the change of temperature, by dividing the moments by the section modulus, the simple expressions, $1\,540\,\frac{d}{f}$ for the crown section, and $1\,265\,\frac{d}{f}$ at the abutment, whatever the span of the arch.

The writer will endeavor to compare the two bridges shown in Fig. 3, as far as the few measurements given will allow. He will assume that the thrust per linear foot may be found by the formula, $\frac{850 \times 128.5^2}{8 \times 25} = 70\,000$ lb. The live load of 100 lb. per sq. ft. distributed over a width of 29 ft. will amount to $\frac{100 \times 29}{17} = 170$ lb. if

distributed over the two arch rings 17 ft. wide; and the bending moment in a three-hinged arch at the quarter span Mr. Mensch.

$$= \frac{170 \times 128.5^2}{64} = 44\,000 \text{ ft-lb. per lin. ft.}$$

The crown thickness of the arch is 22 in., and assuming that the thickness at the quarter point is 33 in., the stresses at the quarter point may be found as follows:

The compressive stress $= \frac{70\,000}{33 \times 12} = 177$ lb. per sq. in.

The stresses in the extreme fibers from the bending moment

$$= \frac{44\,000 \times 12}{12 \frac{33 \times 33}{6}} = 242 \text{ lb.}$$

or a maximum compressive stress of $177 + 242 = 419$ lb.
and a maximum tension of $242 - 177 = 65$ lb.

The average thickness of the arch is $22 + \frac{2}{3} \times 11 = 29.33$ in.

In a fixed arch it will be necessary to assume a greater thickness of the arch at the crown, on account of the temperature stresses; therefore, the thickness at the crown will be assumed to be 28 in. and that at the abutment, 56 in. The maximum bending moment caused by a uniform load, w, per square foot at the crown is about $\frac{w \times l^2}{320}$, and the compressive stress from the thrust $= \frac{70\,000}{12 \times 28} = 208$ lb. per sq. in.; the stresses in the extreme fibers, from the uniform load, $= \frac{170 \times 128.5^2}{320}$, divided by $\frac{12 \times 28^2}{6} = 56$ lb. per sq. in.

As shown before, the temperature stress in the extreme fibers $= 1\,540 \frac{d}{f} = 144$ lb. per sq. in., or a maximum compressive stress of $208 + 56 + 144 = 408$ lb. and a minimum compressive stress of $208 - 200 = 8$ lb. per sq. in. At the abutment, the thrust is $70\,000 \times 1.28 = 89\,500$ lb. and the compressive stress $= \frac{89\,500}{12 \times 56} = 133$ lb. The maximum bending moment from a uniform load

$$= \frac{170 \times 128.5^2}{64} = 44\,000 \text{ ft-lb.}$$

The section modulus at the abutment $= \frac{12 \times 56^2}{6} = 6\,280$ in.^{3}, and the stresses in the extreme fibers $= \frac{44\,000 \times 12}{6\,280} = 84$ lb.

Mr. Mensch.

The temperature stresses $= 1\,265\,\frac{d}{f} = 118$ lb., or the maximum compressive stress $= 133 + 84 + 118 = 335$ lb. per sq. in., and the maximum tensile stress $= 202 - 133 = 69$ lb. per sq. in. The average thickness of the arch $= 28 + \left(\frac{1}{3} \times 28\right) = 37.33$ in.

The stresses from the shortening of the arches will modify the results only very slightly in this case. Although both bridges have about the same stresses, the average quantity of concrete is only 27% greater in the fixed than in the three-hinged arch. Or, assuming that 200 cu. yd. for the three-hinged arch is correct, the saving amounts to only 54 cu. yd., or $675, which is more than counterbalanced by the cost of steel hinges.

The authors state that the slightest movement of the abutments will develop cracks in the arches. This is not the case. A temperature drop of 50° is equivalent to a movement of both abutments of $\frac{1}{3\,640}$ of the span, or, in this case, 0.42 in., and it is a poorly designed abutment in which this movement will be seen. Such a movement would increase the stresses at the crown to $408 + 144 = 552$ lb. in compression and $144 - 8 = 136$ lb. in tension, and at the abutment to $335 + 118 = 453$ lb. in compression and $69 + 118 = 187$ lb. in tension. It is known that ordinary slabs do not crack under stresses less than 300 lb. per sq. in.; hence the factor of safety against the first crack is still ample.

It is true, however, that the temperature stresses and those due to the shortening of the arch would be three or four times as great in the same arch with a rise of only one-tenth of the span, and the advantage of the three-hinged arch in such a case is unquestioned.

In a three-hinged arch, great stresses may be caused by the unequal settlement of the staging, and it has often happened that the hinges have been thrown out of line and out of center, and have had to be dug out and replaced. The failure of a large bridge was due to this cause. This is the main reason that fixed arches are preferred by experienced engineers to the three-hinged type.

It must also be mentioned that three-hinged arches settle considerably more than fixed arches after the centering is struck, and also cause more vibration under moving loads.

In three-hinged arches, the use of deep ribs is a decided saving, which is not the case in all fixed arches. The writer has advocated* the use of I-sections for spans of more than 300 ft., and for spans of from 200 to 300 ft. he advocates T-sections.

The rule given by the authors for finding the center line for a three-hinged arch can be simplified by assuming the entire arch to be loaded by one-half of the uniform load per square foot. This, according to the elastic theory, gives the most favorable line of pressure.

*"The Reinforced Concrete Pocket Book"

AMERICAN SOCIETY OF CIVIL ENGINEERS
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THE EFFECT OF SATURATION ON THE STRENGTH OF CONCRETE.

Discussion.*

By Henry H. Quimby, M. Am. Soc. C. E.

Mr. Quimby.

Henry H. Quimby, M. Am. Soc. C. E.—Mr. Worcester's discussion brings up recollections of some early experience with concrete. Certain 6-in. test cubes made by an inspector in the field yielded very unsatisfactory results when crushed, breaking as low as 500 lb. per sq. in. The cement, which was a standard Portland brand, and the sand, which was New Jersey bank sand, were suspected and retested, but were found to be normal. The concrete was friable—the edges easily rubbed off with the thumb—and it was very absorbent. Tensile test briquettes of the mortar—1 to 3—made at the same time as the cubes, tested as low as 39 lb. at 7 days. Some broken pieces partly immersed in water to observe their absorption were noticed after a few days to have considerably higher tenacity. Similar experiences on other work later, and also some experiments, made it clear that premature setting and drying out prevents concrete from properly ripening.

The trouble appears in hot weather when all the materials have been heated by exposure to the sun, and perhaps when, in addition, a quick-setting cement is used. When concrete is found to be deficient in strength because of premature drying out, continued saturation with water will increase its strength, but will probably not raise it to the normal point unless the saturation be continued for a very long time.

It is evident that concrete wants its moisture and time together.

In order to secure uniform conditions of treatment of test cubes, the rule has been adopted for all inspectors to bury each cube in moist earth as soon as it is removed from the mould, and keep it there until it is sent in to the testing laboratory to be crushed.

*Continued from December, 1913, *Proceedings*.

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MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

By Messrs. L. M. Winsor, H. W. King, and Robert E. Horton.

Mr. Winsor.

L. M. Winsor, Esq.† (by letter).—One of the most crying needs to-day of the entire irrigated West is careful attention to accurate methods of measuring the flow of streams and to the proper distribution of this flow. Through the continual development of larger areas under irrigation, water has become or is rapidly becoming much more valuable than the land it covers. Water users are no longer satisfied with the loose methods of division which were put into practice during the early years of irrigation development. Each season brings new demands for an accurate system of measurement and distribution. For this reason it is high time that the Engineering Profession perfect some accurate, economical, and at the same time simple, device to meet this demand.

The weir in its various forms has been accepted as the device best suited to the needs of the irrigator. Before adopting it as a standard, however, it would seem advisable to make a more careful study of its use, under actual field conditions, than has ever been made. Most of the observations thus far have been conducted under conditions quite foreign to those which are met in the operation of a canal system. The early experimenters used, in the main, larger streams than the ordinary irrigation stream requiring measurements, and the recent experiments were conducted under laboratory conditions which are seldom applicable to the ordinary irrigation stream.

* Continued from December, 1913, *Proceedings*.

† Asst. Irrig. Engr., Office of Experiment Stations, U. S. Dept. of Agriculture, Logan, Utah.

Mr. Winsor. The work being done at the new laboratory at Fort Collins, Colo., under the immediate direction of Professor C. M. Cone, of the Division of Irrigation Investigations, U. S. Department of Agriculture, will without question throw a great deal of new light on the matter of water measurement, and is no doubt the most important work of its kind going on to-day. This laboratory is equipped to make measurements under practical conditions, and the results there obtained will be of untold value to the irrigators of the West, and to the irrigation engineer.

The sharp-crested rectangular weir without end contractions, recommended in the paper, is without doubt one of the best forms, if not the best, now in use, where it can be put in place properly. It is not, however, as easy to erect for ordinary use as is the weir with end contractions. The latter can be placed directly across the channel without the trouble or expense of constructing a channel of approach, the only requisite being the enlargement of the up-stream portion of the stream bed to a sufficient area. In practice, the weir, or any other device, must be adopted by degrees. Farmers or canal operators object at first to any device which is expensive; therefore it is well to give due consideration to one which will give results approaching accuracy, and at the same time admit of construction. When canal operators are educated to the proper point, the most accurate of the various devices may be recommended, even if expensive.

In any case, if the suppressed weir is to be used, it is very important that the area of the up-stream channel be enlarged considerably in order to reduce the velocity above the approach to the weir to such an extent that it will not affect the flow.

The point made by the author that the velocity over a suppressed weir is greater than that over a contracted one is very important when applied to most western streams, which carry a great deal of sediment. This deposit of sediment above the weir is one of the main factors in prohibiting its use under irrigation conditions.

In the writer's opinion, the discussion of the various forms of broad-crested weirs, as applied to the present needs of the country, is less important than the portion devoted to the sharp-crested weir, and particularly to the first three plates or diagrams. Great credit is due the author for his originality in working out this system, and particularly for his clear and concise comparisons between the various experiments.

Mr. King. H. W. KING, M. AM. SOC. C. E. (by letter).—The weir experiments of Francis, Fteley and Stearns, and Bazin have been examined so minutely, and have been grouped and studied so carefully and thoroughly by so many able hydraulicians that it would hardly seem possible to evolve anything really new from them. The author, however, by his method of combining into working diagrams the results

of these experiments and the more recent ones of the Cornell and University of Utah Hydraulic Laboratories, has done much to simplify the use of this mass of data. The contribution is a most valuable one, because of the new methods of analysis developed, as well as the practical value of the results obtained. Mr. King.

Inasmuch as the author did not endeavor to derive a general formula of discharge, he was able to obtain several formulas for each series of observations which fit very closely each value given. These exact formulas were in turn used in deriving the quantities used in the diagrams. It is evident that the author is consistent in his methods of constructing these diagrams, and they represent accurately the results obtained from the experiments. There can be no doubt that quantities can be picked off the diagrams with as great accuracy as is justified by the results of the original experiments, and, doubtless, they furnish the most ready and reliable means yet provided for solving problems of the type to which they apply.

This paper, however, should be considered as a very comprehensive study of one phase of the subject of weirs rather than a general analysis. The author takes up the single case of a weir in a channel of rectangular cross-section in which the length of the weir is equal to the width of the channel. This is the condition under which many, if not most, of the weir experiments above referred to were made. In constructing a weir for the measurement of water, it frequently happens that these requirements may be fulfilled. It is not true, however, that this may be done in all cases. In many cases the engineer wishes to measure the flow of water by the weir method in a canal of trapezoidal section or a stream of irregular section. There is, moreover, a question as to whether it will be practicable under all conditions to require the measurement of irrigation waters by suppressed weirs in rectangular channels, as the author suggests. These ideal conditions should be obtained when practicable, but there seems to be no good reason for limiting the use of the weir to this one case.

Occasions will continually arise when it will be desirable to construct weirs across other than rectangular channels, and there will consequently continue to be need for an accurate solution of these problems. It also appears that we are not yet ready to dispense with the weir with end contractions, and, therefore, there is still need for a means of determining the discharge over such weirs. It is doubtless true that, for maximum accuracy, with our present limited data and knowledge regarding the flow over weirs, the channel of rectangular section and suppressed weir should be used where practicable. This, however, is due to the fact that the experimental data thus far obtained apply more particularly to weirs of this type, and not

Mr. King. because those constructed under other conditions may not be possible of as exact solution.

In order that a method of solution of weir problems may be of general application, it must provide for a channel of approach of any cross-section. It should also, with proper modifications, include the weir with end contractions. It would appear, therefore, that the foundation of the method of solution must apply to the weir discharging from the theoretically quiet pond. The coefficient of discharge under such conditions is well known to be very close to a constant quantity within the range of head covered by our present experimental knowledge. The deviation of this coefficient from a constant, therefore, is due almost entirely to effects of velocity of approach. The present weir formulas, with the exception of Bazin's, use this general method of solution; that is, they determine the discharge from a still pond and correct the observed head, so as to include the increased discharge due to velocity of approach. That these formulas do not give results of sufficient accuracy is unquestioned. The trouble, however, does not lie in erroneous theory or assumptions, but in being based on insufficient if not partly unreliable experimental data.

There still remains to the Engineering Profession the problem of providing a satisfactory general method of solution for all weir problems. Such a method of solution must be based largely on new data, which should cover a wider range than those existing, and be of unquestioned accuracy. New experiments are needed to give more definite knowledge of the discharge over weirs without velocity of approach, which will replace the assortment of formulas in use at present. There is even greater need of complete data which will give a clear understanding of the laws governing the flow of water, the distribution of velocities in the channel of approach, and the effect of these laws on the discharge over weirs. At present, adequate facilities for such experiments are not available, but it is to be hoped that a means will soon be provided for obtaining these much needed data. With them a ready means should be provided for solving all weir problems within the required limits of accuracy.

Mr. Horton. ROBERT E. HORTON, M. AM. SOC. C. E. (by letter).—The writer regrets that he has been unable to find time to give this paper the careful perusal it undoubtedly deserves.

Heretofore, a thin-edged weir, constructed substantially in accordance with the rules laid down by the late James B. Francis, Past-President, Am. Soc. C. E., has commonly been designated a "standard weir", and has often been accepted by the Courts as a correct method of measuring water, whether or not it was so in fact. The writer agrees with Mr. Lyman to the extent that a suppressed weir is an equally good "standard", but questions the propriety of legalizing this

or any other form of weir as a "standard". It seems to the writer that there are now a considerable number of forms of weirs, all sufficiently well calibrated to serve as excellent standards in particular cases, no one of which is the best standard for all cases. Mr. Horton.

The hydraulics of weir discharge is by no means as simple a matter as it may appear to a neophyte. A good engineer can get good results with good weirs of various forms. The writer does not hesitate to say that he has seen "Francis Standard" weirs abused—and the results sworn to in Court—in a startling manner. Such abuses are usually coupled with the opinion, tacit at least, that as the weir is denominated "standard", it is consequently fool-proof, and the element of personal equation is eliminated.

Great injury has been done by writers of textbooks on hydraulics by placing undue stress on the merits of so-called standard weirs and orifices, especially the latter, as compared with other forms. Almost any kind of a round-edged orifice, with a reasonable edge radius, is subject to less variation in coefficient in practice, and is less likely to be obstructed or change its form, than a thin-edged orifice. The same is true to some extent for weirs. It is also true in both cases that the round-edged form discharges more water for an opening of a given size then the thin-edged form. This is a matter of some importance, in the case of orifices, in measuring the supply of water-wheels through flume or bulkhead openings where water is sold or leased in square inches or other units. It is a matter of great importance in designing flood spillways for dams, because, with a given length, the round-crested spillway, having a larger coefficient, will discharge a flood with a less depth of back-water than a broad flat or a thin-edged weir. The floods in the spring of 1913 demonstrated pretty clearly that spillways of dams form the best practicable method of measuring the maximum flood discharge of rivers, and for that purpose, ogee or round-crested weir sections are the best practicable standard.

Mr. Lyman describes various base formulas which, with suitable coefficients, have been used for calculating the discharge over weirs of various forms. In the original deduction of the U. S. Geological Survey experiments of 1903*, the writer chose the theoretical formula, which had also been adopted as a base formula by Francis for thin-edged weirs:

$$Q = C\,L\,H^{\frac{3}{2}}$$

The reasons were as follows:

(1) The results in this form for irregular sections are directly comparable with the discharge of a thin-edged weir, when the latter is calculated by the formula then in most common use.

*As described in Water Supply Papers Nos. 150 and 200. The table of multipliers appearing at the end of these papers was inserted after the paper left the writer's hands, and without his sanction.

Mr. Horton.

(2) The coefficients for many forms of weirs of irregular section are not continuous functions of the head, so that the full history of the discharge under a wide range of heads cannot be represented by any single formula with a constant coefficient.*

(3) If, as is true in many, but by no means all, cases, a variable coefficient must be used in formulas for such weirs, then it seems better to throw all the variation into the coefficient and make the computation as simple as possible by using the $\frac{3}{2}$ power of the head, instead of causing the head to vary according to some other fractional or decimal power, which would require the use of logarithms, logarithmic paper, or a logarithmic slide-rule for practical computations.

Judgment based on experience is one of the best checks on computations, and is often a valuable criterion as to the reliability of experimental results, and, for this reason, the writer prefers to compute experimental results in the first instance in some manner that will make such results commensurate with some established scale or "yard-stick", as the Francis formula for weirs, or the Chezy-Kutter formula for channels.

If the results appear reasonable, they may afterward be applied to any formula desired. Francis applied an exponential formula to the Merrimac River Dam.

The writer early recognized the value of the exponential formula for certain forms of weirs, and applied it where it seemed to fit. Later, Mr. Philip Parker, using the writer's computation of the Bazin, U. S. Deep Waterways, and U. S. Geological Survey experiments, deduced exponential formulas for a large number of weirs.†

The writer is not in sympathy with the practice adopted by some, of working out a table of multipliers to be applied to the calculated discharge over a thin-edged weir to obtain the discharge over an irregular crest-section. First, the discharge over the weir in question might as well be calculated in the first instance, thus saving a deal of computation, with chances for error; second, for a given head and cross-section, the velocity of approach correction varies with the discharge coefficient, so that for two weirs, one thin-edged and the other of irregular section, with the same measured head, but with different sections of approach, the use of multipliers becomes confusing and may be a source of error.

The use of either the Francis base formula or the exponential formula, both with varying coefficients where necessary, seems to the

* The presence of *points d'arret*, cusps, etc., in weir coefficient curves is shown very clearly in some of Bazin's experiments. See the writer's discussion of the paper by the late George W. Rafter, M. Am. Soc. C. E., entitled "On the Flow of Water Over Dams". *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 345.

† A few of Mr. Parker's formulas appear in his recent work "The Control of Water."

writer better practice, for cases where there is no abrupt change in form of nappe, as when the water breaks free from a flat crest and assumes the thin-edge weir form, the exponential form of expression probably gives in many cases the simplest and most complete history of the discharge. Mr. Horton.

In conclusion the writer is prompted to remark, regretfully, as regards hydraulic science, "Of the making of many formulas there is no end."

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TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.

Discussion.*

By N. T. Blackburn, Assoc. M. Am. Soc. C. E.

Mr. Blackburn.

N. T. Blackburn, Assoc. M. Am. Soc. C. E. (by letter).—The writer was introduced to the practical side of the stadia method of surveying immediately after graduation from college, and has had a good deal of experience in its use along the coast of Texas, under the U. S. Engineer Office. Its adaptability and flexibility are so great that he has used it almost entirely, except for hydrographic surveys which take in large areas some distance from shore. Where two transits or sextants are not available for locating lines of soundings by intersection, and the lines do not run too far out from shore, a very good survey can be made by locating points on the lines of soundings with a stadia rod held in the sounding boat.

The writer agrees with the author as to the use of fixed stadia wires and the disregard of the $f + c$ factor. Where carefully measured base lines are run by this method, taking short distances, when the atmospheric conditions are favorable, and with great care in reading the rod, its use may be advisable. For ordinary work the $f + c$ factor is certainly a useless refinement, particularly on side shots. The writer has never attempted to read the rod closer than the nearest 5 ft. on side shots of any length, and he has never been able to guarantee that his side shots in open country, in the middle of the day, for distances of more than 1 500 ft., were within 15 ft. of the correct distance, on account of the trembling of the air. With ideal

* Continued from December, 1913, *Proceedings*.

Mr. Blackburn.

weather conditions—a cool, cloudy day with no wind, or very light wind—and short shots, it may be possible to read the rod close enough to allow a correction for $f + c$; but the writer's experience has been that it is generally impossible even to graduate a rod to fit the instrument exactly the same at two different times in the day.

On his first work the writer used the ordinary diamond diagram for the stadia rods, Fig. 2. This marking is plain and easily read except that in the hot sun in open country and at distances of 1 000 ft. or more, the red triangles tend to blur, and the dividing line is hard to distinguish. Moreover, any rod which has the same diagram for each 1-ft. interval is confusing to read. The red figures, however, are an advantage in woods or brush, as they show up in contrast to the foliage and undergrowth. A better form of marking is shown by Fig. 3. This was given to the writer by a member of this Society who was connected with the U. S. Reclamation Service at one time, and is understood to have been used by that organization. It will be noted that only every fourth foot is graduated to tenths. One stadia wire is set on a dividing line between foot marks so that the other wire falls within the divided block. Distances can be thus read more rapidly, particularly for long shots, as the length subtended between stadia wires can be read in 400-ft. blocks.

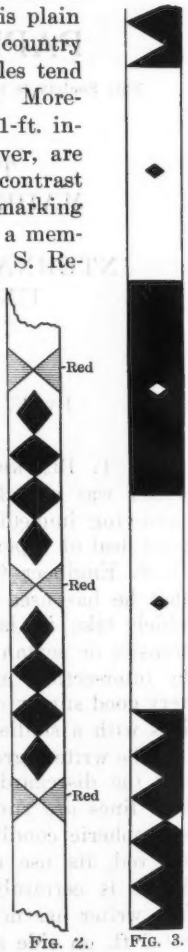

FIG. 2. FIG. 3.

A rib on the back of a rod is useful to prevent warping and to furnish a handle by which to hold it, but it adds to its weight and complicates its use as a line rod when taking azimuth. It has been the writer's experience that stadia rods warp badly unless carefully watched.

The author's tables showing errors of closure for stadia traverses are interesting and instructive. On a survey of Buffalo Bayou, Texas, a base line was run along each bank by transit and stadia, and the two lines were tied across at intervals of from 1 to 3 miles, thus forming a series of closed traverses from which the errors of closure could be computed. About six of these closures were thus made, and it was found that the error varied from 1 in 155 to 1 in 2 200. The distances on the 1 in 155 traverse were afterward chained and the error reduced to 1 in 1 930. The stadia work was done very carefully, and all distances were read forward and back, but no $f + c$ correction was made.

Mr. Blackburn.

In keeping notes the writer has always numbered consecutively his stadia hubs, and also all side shots from each hub. Then any side shot is located by a double number, first, that of the stadia hub from which it was taken, and then that of the side shot, thus: 15-1, 15-2, etc. A page of notes would appear as shown by Table 24. True south is taken as zero and true north would then be 180 degrees.

On the right page is made a rough sketch of the area surveyed, and the side shots are spotted thereon with their number placed alongside. A note is also made, on the same line as the notes, as to the object on which the side shot was taken. This method has always been clear to the draftsman. As a usual thing, it is better for each man to plot his own work, but this is not always possible, particularly if it is to be plotted as fast as taken. Where the rodman remains always some distance from the instrumentman, and is able to sketch fairly well, it is usually a good practice to furnish him with a book in which to sketch the topography and locate the points on which he has held the rod.

TABLE 24.

At.	On.	Distance, in feet.	Azimuth.	Magnetic bearing.	Vertical angle.	Elevation.
15	14	563	12° 20′	S. 4° 30′ W.	.....	.....
(H I. = 15.4)	16	750	220° 49′	N. 32°-50′ E.	.....	.....
.....	1	530	270° 50′	.....	5° 10′	.....
.....	2	650	155° 13′	.....	.....	.....
16	15	750	40° 49′	S. 32° 50′ W.	.....	.....
(H I. = 17.2)	17	920	187° 15′	N. 0° 45′ W.	.....	.....
.....	1	220	192° 00′	.....	.....	.....

The writer has always used with success the method of observation on Polaris for azimuth given in Johnson's "Surveying."* However, it is plausible that the heat of the light used to read the verniers might affect the instrument, and he is glad to know of this.

During the winter of 1911-12 the writer organized and supervised a topographical survey of the Guadalupe River, Texas, on which methods similar to those described by the author were used, although not so accurately in some particulars. This survey extended over about 38 miles of the river, from a point about 14 miles above the mouth to the Town of Victoria (Mile 52), and covered not only the bed and

* Page 558 *et seq.*, 15th Edition.

Mr. Blackburn.

banks of the river, but an area about ½ mile back on each side. All distances were measured by stadia. A base line was run along the bank of the river, crossing from one side to the other wherever necessary to get the most open country. On this base a double-rodded line of levels was run very carefully. This base line then formed the horizontal and vertical control for the entire survey. The meanders of the river and all topography between the water's edge and the top of the bank were taken directly from this line. The topography back from the river bank was obtained by running random lines from points along the base. These were similar to the banco traverses described by the author. Most of the elevations for topography were obtained by level, running alongside the transit. The transitman located the controlling points, and the levelman obtained the elevation, which was immediately given to the transitman and entered in his notes. This method requires more instrumentmen, but it is very rapid and simple, and saves much time in the reduction of stadia notes. Where it would have required several "set ups" of the level to obtain the elevation, resort was occasionally had to vertical angles. The true meridian was obtained from a magnetic base established by the U. S. Coast and Geodetic Survey at Victoria, and was checked at a place about 14 miles below the starting point by direct observation on Polaris by the method previously referred to. The field work of this survey cost about $125 per sq. mile. This high cost was due to the bad weather encountered and to the thoroughness with which the work along the river was done. Much of that work has not been described herein. The cost includes also the purchase of considerable equipment, fitting up a houseboat, etc. The survey was made for the purpose of preparing an estimate and report on the improvement of the river by locks and dams. For the area back of the river bank the cost per square mile was probably less than one-half the figure given.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

BY MESSRS. HIRAM F. MILLS AND T. U. TAYLOR.

Mr. Mills.

HIRAM F. MILLS, HON. M. AM. SOC. C. E. (by letter).—The writer is unable at present to make a critical study of this subject, but would like to ask Mr. Hazen if he has sufficiently provided for the condition —observed many times in New England rivers—that a very small run-off is often followed by a second year of small rainfall, which gives more trouble because following after ponds and ground-water were lowered?

Exceptionally, we have the third very dry year, as in 1911, when storage reservoirs were generally empty, and very great anxiety was relieved by a moderate rain in October and more in November. This third year, of course, he would regard as the exceptional year, when emergency supplies or borrowing may have to be resorted to; but the writer is not quite sure that the second very dry year—which often occurs—is provided for.

Mr. Taylor.

T. U. TAYLOR, M. AM. SOC. C. E.—Among the elements or factors entering into the problem of the storage of water for municipal water supply, the author mentions the following:

1.—The size of catchment area or water-shed;
2.—The mean annual run-off per square mile;
3.—The portion of water area and the loss by evaporation from it;
4.—The natural storage in lakes, or deposits in sands, gravels, or pervious materials;
5.—The regularity of flow.

* This discussion (of the paper by Allen Hazen, M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, and presented at the meeting of December 17th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Taylor.

Having spent more than half a century in the West, and having been more than half that time a close student and observer of water conditions in the great southwest section of the United States, the speaker welcomes any discussion of storage problems, even though the data for the most part refer to sections of the country with conditions unlike those of his own. Such discussions are valuable, although they may serve the sole purpose of convincing engineers that the data for the East cannot be applied to the West, and that the man who does this brings disaster on the enterprise and should bring it on himself. However, such discussions are valuable, whether for municipalities, irrigation, or power. In the Southwest, especially in Texas, all these problems have to be considered, and it sometimes happens that two, or perhaps all three, storage problems may occur on the same river.

Two of the elements entering into the problem, as mentioned by Mr. Hazen, can be omitted in Western or Southwestern problems, namely, the water area in the water-shed and the natural storage in lakes. The three fundamental factors in Texas are:

1.—Area of water-shed;
2.—Regularity (or irregularity) of flow;
3.—Run-off.

The area of the water-shed can be ascertained with a reasonable degree of accuracy, and the regularity of flow can, in general terms, be ascertained by local testimony—and it is about the only factor in the problem where the testimony of the oldest inhabitant is worthy of serious consideration. For accurate calculations, however, it would be necessary to have hydrographs constructed from daily records of flow, and these can only be obtained by gauging stations established and maintained with unbroken records of gauge readings and of annual rating curves. The third factor, the run-off, is the one which contains possibilities of the gravest error. Any attempt to apply run-off data obtained in one section of the country to a locality or stream in another section is likely to result in great damage, unless Providence is kind. It is full of the utmost danger, as many wrecks in the West will bear silent and bitter testimony. The regularity of flow, the quantity of daily flow, the mean monthly flow, the mean annual flow, and the run-off are data that can be obtained only by careful observations and measurements for the locality, station, and stream concerned. The run-off for the same stream, at different stations along its course will be found to vary greatly. The speaker calls to mind a case where the run-off on a stream at one point is 0.1 sec-ft. per sq. mile of water-shed, and at a point 1 mile above it is only 0.01 sec-ft. A stream may course its way through a coastal plain of a reasonably flat country, then enter a canyon section where

floods occur with torrential violence, and then again, in its upper reaches, it may enter into a flat upland section. It will be found, and, in fact, has been found, that the run-off and flow in second-feet per square mile of water-shed differ greatly. The problems in the West are largely concerned with the rainfall, run-off, mean flow, and the necessary storage capacity to equalize the flow or to equalize it sufficiently for the municipality, irrigation project, or power problem. Mr. Taylor.

Careful daily records and observations on the Colorado River of Texas, at Austin, Tex., for the last 14 years, have shown that the average or mean flow for this period has been 1 820 sec-ft. The water-shed of 37 000 sq. miles would give a mean flow of $\frac{1}{20}$ sec-ft. per sq. mile. The lowest mean annual flow for this period was $\frac{1}{74}$ sec-ft. per sq. mile, and the largest flow on record, occurring at the time the Austin Dam failed, April 7th, 1900, amounted to less than 4 sec-ft. per sq. mile. The lowest flow on record occurred from August 15th to 20th, 1910, when the run-off was about $\frac{1}{1800}$ sec-ft. per sq. mile.

The mean annual rainfall on the water-shed of the Colorado above Austin for the period was practically 24 in., but the mean flow for the period was only 1 820 sec-ft., which is equivalent to a run-off of 2.88%, or 0.7 in.

With these staggering facts available, a $4 000 000 enterprise was established in a neighboring territory, when, after unusual rains, the engineers awoke to the fact that, in using an assumed run-off derived for other regions, they had omitted the most vital factor, and had, according to their own admissions, over-estimated the possibilities of the scheme by a large percentage. The money had been spent, the water-shed was there, and the rains came, but the run-off refused to run with the vigor that it was assumed to display on such occasions. The only plea of defense is that the engineers were misled, as to the run-off, by a book on irrigation, when the book made no statement as to the run-off in the territory mentioned.

A large municipality authorized the construction of an impounding reservoir and its appurtenances at a cost of $1 000 000, on an assumption of a run-off of 19%, and again there was a failure to come up to expectations.

The run-off is a complex factor, depending on topography, vegetation, kinds of soil (whether cultivated or uncultivated), rainfall, distribution of rainfall as to time of year and as to growth of vegetation, and, what is still more vital, the condition of the soil at the time of the rains. Personal observation has convinced the speaker that a 2-in. rainfall in 24 hours may at one time give a run-off of 25% and at another time no run-off whatever. In 1910 there was the lowest flow of the Colorado River at Austin, but it was not the year of least rainfall, nor the year of lowest mean annual flow. Each stream is a

Mr. Taylor.

problem unto itself. An experienced engineer will go very slowly in such matters. Like Davy Crockett, he will be sure he is right, and then go ahead, and not, like many, "rush in where angels fear to tread."

The Colorado of Texas has a canyon section or power section, a municipal section, and an irrigation section. At present, due to a lack of storage, neither the power section nor the irrigation section is reliable as a revenue producer, as the flow is spasmodic and irregular. The construction of storage reservoirs on this river is a simple and entirely feasible problem, and it would be an easy matter to store about 20 000 000 000 cu. ft.; and this, in the driest years, would give a minimum flow of 800 sec-ft., which, with the total head of 240 ft. that could be obtained by the various dams, would give about 20 000 h.p. Then, in addition, this reliable and equalized flow would insure the irrigated rice crop 250 miles below, near the coast. Altogether, the Colorado offers one of the best problems for storage known to the speaker. Its records as to flow, area of water-shed, run-off, dam sites, and the demand for power and for reliable flow for irrigation, have all been worked out, and there need be no assumptions as to any factor.

The method suggested by Mr. Hazen for rating the dry years is safe, and would fit any section of the country, because each river would be writing its own biography. It might sound a little more comforting to the people of the West if the percentages were given in "wetness" instead of "dryness". The dryness feature is obtruded on them with sufficient emphasis already, because it is a land

"Where the prairie dog kneels,
On the back of his heels,
And silently prays for rain."

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THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.

Discussion.*

By Messrs. W. J. Wilgus, J. E. Willoughby, Frank C. Boes, Allen Hazen, H. C. Vensano, Leonard Metcalf, William B. Jackson, F. Lavis, Alexander C. Humphreys, Henry Floy, Clinton S. Burns, Stuart K. Knox, and J. H. Gandolfo.

Mr. Wilgus.

W. J. Wilgus, M. Am. Soc. C. E. (by letter).—The distinction made by Mr. Alvord, that the formal setting aside and labeling of depreciation reserves should entitle the owner of a property to a return on the full cost of its reproduction new, and that the non-observance of this technicality should deprive him of that right, does not appeal to the writer as being just or logical. Surely, if the general balance sheet of a corporation shows an excess of assets over liabilities sufficient to offset the accrued depreciation of the property, there would seem to be no more reason why the return should be based on an impaired investment than that the rent paid by a tenant should be lessened if the owner, who has agreed to keep the premises in repair, elects not to maintain a special fund therefor in the bank.

The majority of railroads have ample profit and loss surplus accounts invested in outside securities or in the property in excess of the amount on which a fair return is earned; and this surplus, for all practical purposes, is a depreciation reserve which insures the ability of the corporation to meet its liability of paying for maintenance out of income. A failure by a corporation properly to meet its liability

*This discussion of the paper by John W. Alvord, M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, and presented at the meeting of December 17th, 1913, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Wilgus.

of maintaining the property out of the rate, is usually the result of over-payment of dividends, and eventually brings its own punishment through the necessity of a reduction or suspension of dividends during the period of rehabilitation. This course is obligatory, as charges for rehabilitation legally can only be made through income.

Whether or not a depreciation fund is actually set aside would seem to be immaterial. What the public is interested in is the proper upkeep of the property without an increase, for that purpose, of the capital burden which it is expected to bear through the rate.

As Mr. Alvord states, "* * * no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand"; but to determine whether or not it is in hand may be done just as effectively through an examination of the balance sheet in conjunction with a valuation of the assets, as in looking for a labeled fund in the bank. To "fine" the owner who elects to protect his liability in some other form than by means of a special fund, seems to be rather a drastic punishment for his selection of one of two alternative treatments of an accounting detail.

Mr. Willoughby.

J. E. WILLOUGHBY, M. AM. SOC. C. E. (by letter).—Few engineers will disagree with Mr. Alvord's suggestion,

"That if a sinking fund * * * for depreciation is actually kept * * * in trust as part of the property, it should * * * receive the same rate of fair return as the remainder of the property 'used and useful for the public.'"

The writer, however, believes that this suggested addition to the capital account will find little favor. Depreciation is an operating expense when applied to the accounting of a going public utility, and there is no more reason for capitalizing the depreciation item of operating expenses than for capitalizing any other single item, for instance, the wages paid to enginemen in charge of railway trains.

Furthermore, such funds have not been generally provided as a part of the capital account of our public utilities, and we gain directness by discussing the actual conditions, and the need of providing for depreciation as an operating expense, so that the accounting will always show the value of the public utility for any purpose, rate-making or otherwise.

The accounting of a public utility covers other things than the cost of the plant, and it is the ultimate finding of the accounting (exclusive of the speculative and intangible features) that fixes the value of the utility. The author, however, limits his discussion to that "line of evidence as to value known as the reproduction method, or cost new less depreciation." By thus disregarding all other elements of value that enter into a public utility property, the author has no credit account to offset depreciation, except the cost of reproduction

Mr. Willoughby.

new of the physical property of the utility, and he would limit the rates to be charged by the utility corporation to a reasonable return on the capital necessarily expended for a new plant less the accrued depreciation. This means that the utility corporation must, immediately after reproducing its property new, write off as loss the amount of the accrued depreciation, although that accrued depreciation has actually resulted from service rendered to the public, and, although the only practical way of reproducing the physical property will be to invest first capital to the extent necessary to reproduce the property new.

It is generally accepted that the reproduction of the physical property of a public utility which has been in operation for some years so as to show its exact stages of depreciation would be at a greater cost than to reproduce it with all new materials. Consider the difficulty and expense of providing and assembling into a completed structure the curve-worn rails, the partly decayed ties, the partly fouled ballast, the partly failed bridges, trestles, and culverts, and the argument is completed. Consider then the injustice of the suggestion of the author that the property owner should be penalized because of the impossibility of assembling his plant, except as a structure composed of new elements. What the public pays for is service. If the service is satisfactorily performed, as in the railway after some years of operation, over rails, cross-ties, and ballast, each exhibiting but 55% of their cost new, less salvage, and if to perform that service it has been necessary for the owner to expend in the past 100% for those items, the owner should receive a reasonable return on that 100 per cent. The public's protection lies in scanning the operating expenses to see if the charge for maintaining the property is unduly high, as a result of the failure of the railway company to make provision in the past for the depreciation at the time the depreciation actually occurred, and not to deduct from the capital originally expended the amount of the accrued depreciation. It is only after the operating expenses are deducted that any return can be made to the owners of the stocks and bonds. If the operating expenses are unduly high, on account of failure to make provision for the depreciation as it had accrued in the past, the penalizing of the owner occurs in the denial of deduction from the gross earnings of so much of the alleged operating expenses as is due to the failure of the owner to provide for the depreciation at the time of its accruing. The method of penalizing the owner, as suggested by the author—the deduction of the accrued depreciation from the reproduction cost new—is a confiscation of property to the amount of the accrued depreciation, because the amount of the accrued depreciation was originally and necessarily put into the property before any depreciation could take place, and whatever has taken place

Mr. Willoughby.

is properly to be regarded as an expense of operation. No engineer would deduct from the physical elements of a property the operating losses due to bad financial arrangements, extravagance, or incompetency.

To illustrate further: take a cross-tie costing 80 cents in place, and lasting 8 years. For each year following the placing of the tie, there should be charged out as operating expense the sum of 10 cents (less by such interest as it may earn), as the proportion of the cross-tie consumed in that year's operation, so that when the cross-tie is replaced in the eighth year only 10 cents of its replacement cost would be charged into operating expenses, the other 70 cents being taken from the depreciation fund. The purpose of the depreciation fund is to distribute the operating cost of renewal over the number of years that has been taken to consume the tie. Where a depreciation fund has not been accumulated to offset the actual depreciation, the monetary value of such depreciation will go into the profit and loss account as a debit, but there may be such values in that account—like accumulated reserves—which will entirely absorb the debit. Depreciation, when regarded as an operating expense, can never be used to lessen the reproduction cost of a public utility. It will always affect the value of the utility as a going concern.

Every public utility should make provision in its accounting for depreciation, because such an account is necessary to show the annual operating cost of any plant, and to reflect in the accounting the financial and physical condition and value of the plant. It should cover "all the losses of value that occur in the property, plants, or parts thereof, from wear and tear, obsolescence, or inadequacy."

For a public utility that has permitted the depreciation to accumulate without providing for an actual fund to offset the same, then the amount of such accrued depreciation (in such event a debit) will go into the profit and loss account, there to be offset by the credit items of other accounts. Correct accounting does not require the actual presence of cash, except as to the amount of the item of cash on hand.

As to what rates are necessary for a fair return for any public utility, if confiscation is not to result to the property in the legal sense: the suggestion is made that the rates should be sufficient to provide for:

1st.—The payment of taxes and other public dues;

2d.—The payment of all operating expenses, including as a part thereof a charge sufficient to care for the annual depreciation that takes place;

3d.—The payment of a reasonable commercial interest (commercial interest being one that considers and includes the element of risk involved) on the reproduction cost new of the property, including the necessary working capital;

Mr. Willoughby.

4th.—The payment of a like interest on the intangible values of the property and the unearned increment;

5th.—And, finally, the accumulation of funds sufficient to provide for the non-earning additions demanded by the public. The commercial interest earning additions and betterments can be capitalized and becomes a self-supporting part of the cost of the plant.

Mr. Boes.

FRANK C. BOES, ASSOC. M. AM. SOC. C. E. (by letter).—After reading Mr. Alvord's paper, the writer tried to picture to himself some of the possible results of putting such a system of rate-fixing into general operation.

Consider two adjoining and otherwise similar communities served by two power companies, the one having an old and decrepit plant, but still capable of rendering efficient service, and the other having a new and up-to-date plant. The large users of service, of course, would gravitate toward the community served by the depreciated plant, thereby decreasing property values and rents in the community which they left, and making it impossible for the new plant to compete with the old one. The continual see-sawing of rates, due to periodic repairs and renewals, would make it impossible for service rates to reach a standard level, thus making it impossible for a consumer to foretell what his next year's bill for service would amount to, and would also cause much litigation and confusion.

To fix rates purely on a basis of physical value, would remove incentives for good management and economy, and put a premium on wastefulness, expensive plant, and unnecessary repairs and renewals. Over-expenditure on plant, for the sake of keeping the rate up, is more to be feared than a reasonable amount of decrepitude, for, after all, the service rendered is what is paid for.

If such a system were in vogue now, those who live on Staten Island would be paid a substantial sum for riding on trolley cars which have long outlived their years of usefulness and are run by a plant which is in the condition of the "one-hoss-chaise." They are consoling themselves by the thought that they pay, not toward earnings on a dilapidated outfit, but for the service rendered—a 5-mile ride. A company owning a modern plant might give worse service.

Mr. Alvord states that the average life of the component parts should give the composite life of the plant. This must be an oversight on his part, for, assume a plant to consist of two parts, A, costing $10 000 and having a life of 5 years, and B, costing $5 000 and having a life of 20 years. Surely the composite life of the plant is less than 12½ years, for its major part is renewed twice in that time.

In fixing rates by such a method, one must also consider that the depreciation curve is not a straight line. It is permissible to establish

Mr. Boes.

a depreciation fund by the straight-line formula, for that is merely a convenient method by which the owner may distribute his renewal costs evenly. The amount of the depreciation fund by no means represents the actual physical depreciation, and to base the deduction of rates on an amount which is taken arbitrarily, and is greater than the actual depreciation, would be unjust, particularly if the service were still unimpaired. When a laborer reaches old age, his earning power is greatly reduced, but is he worth less at thirty-five than at twenty? One cannot expect to deduct $3 000 from a $10 000 plant, in fixing rates, and still have a $10 000 service maintained. If the returns are not based on the entire investment continuously, the investment will not have renewed itself at the end of the plant's term of life, and the owner will not have the plant that he had to begin with. Depreciation is merely another form of capital, and must be kept continuously earning returns on its entire amount.

It seems unjust to expect to pay less for service in the fifth year, the service being exactly the same, merely because some parts of the plant will require replacement at a date then nearer than in the first year. Does one pay less for milk given by old cows or for apples grown on old trees?

Mr. Alvord intimates that the reduction in rates should be considered in the nature of a fine, but surely it would be unjust to have a fine imposed, if, when the time for renewals actually arrived, the owner produced the funds and put the plant in shape. What then? Would the amount of the fine which he had paid be returned to him? A man cannot be fined because it is thought that he is going to commit a crime.

An argument sometimes advanced, and by the Courts, is that one should not make one valuation for rate-making purposes, and another valuation for the purpose of sale or purchase; that is, if valuation is deducted in making a sale of a plant, it should be deducted in making the rates of service. Consider some private business: If an owner wishes to sell his factory, he must certainly make a deduction for depreciation, but the price of his merchandise is not affected in any way by the age of his factory or by the condition of his plant. Neither do his customers inquire as to whether or not he has an adequate renewal fund actually in hand. That responsibility rests with himself alone.

Though there is a difference between fixing the prices in a private business, and fixing the rates of a public utility which enjoys monopolistic privileges granted by the public, the fundamental principles remain the same in each case.

The approaching repairs and renewals are an obligation and a cause of worry to the owner, and to him alone. He has to pay for them, and

the consumer is in no way concerned with it, as long as the service remains the same. The consumer is not concerned with how near the day of collapse the plant may be, as long as the collapse never occurs. The consumer is concerned only with impairment of service. When that occurs, then a fine may reasonably be imposed. Mr. Boes.

In fixing rates, one need only be sure that the property is in satisfactory working order and that proper service is being given. The owners can then be left to worry about the depreciation for themselves.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—Depreciation has to be taken into account in both the operation and valuation of public service properties. It has to be taken into account because it is one of the fundamental facts that cannot be ignored. Mr. Hazen.

The writer is familiar with depreciation, especially in water-works properties, and what he has to say relates primarily to them.

Allowances for depreciation have actually been made in nearly all properties of this kind. This is not the less true because the amount allowed has not been determined in a rational manner, and frequently has not even been called depreciation. In most public works the first cost has been paid by money raised by the sale of bonds, and a sinking fund has been established, and annual contributions made to it, intended to be sufficient to provide for the payment of the bonds when they became due. This, in reality, providing for depreciation, even though the word "depreciation" is not used.

Another way of providing for depreciation which has been common, has been to pay for new construction out of earnings. This also is an efficient means of allowing for depreciation, although the methods and amounts to make it efficient have seldom been carefully thought out. In general, with publicly owned properties, the allowances that have actually been made for depreciation in these ways have been above the truth. This has been fortunate, as the excess has represented only a slight addition to rates which have not been burdensome, and the resulting strong financial condition of the works has tended to good service, worth more than its cost to the takers.

In the relatively small number of cases where no allowance, or an inadequate allowance, for depreciation has been made, there results sooner or later a cramped financial condition tending to bad service, and equally unfortunate for the works and for the takers.

The measure of depreciation for any period is the decrease in the fair value of the property under discussion, or any part thereof. In valuation proceedings, where the cost of reproduction is being considered, the decrease in the value of the property, as compared with the value of corresponding new property, is the measure of depreciation. The fair value of the property is to be considered always as the fair value between a willing seller and a willing buyer. To take

Mr. Hazen.

a concrete case, it may be considered how much less a willing buyer would probably pay and a willing seller take because the pipes in the streets had been in use for a time, and had suffered the depreciation naturally incident to use. Obviously, some deduction would be made. The carrying capacity of the pipes is less than that of new pipes, owing to rusting and tuberculation; there is more leakage from the joints in old pipes than in new ones, representing loss of water and profits. It may be presumed that the pipes in an existing system laid from time to time are perhaps less efficiently arranged as to sizes and locations than would be the case in such a pipe system as the buyer would lay for himself. These would probably be the most important points to be considered as between the buyer and the seller in deciding what deduction was to be made. In addition, any peculiar conditions of any kind affecting the value of the property would be duly considered.

The average age of pipes in any system in a growing city will ordinarily be from one-third to one-half of the age of the system, and seldom more than 15 or 20 years. The bare fact that an average of 15 years has elapsed in a possible ultimate life of from 75 to 150 years, which is assumed by the author, would have but little influence between a willing buyer and a willing seller.

The case is somewhat different with boilers, which may be assumed to have a life of 20 years, or some other definite period, and will then have to be replaced. The boiler represents the exceptional case in water-works affairs, and not the ordinary one.

The writer believes that there is no such thing as a period that can be called the life of cast-iron pipe. Different pieces of pipe may have useful lives ranging from a few hours to periods so long that they extend beyond our experience. Any statistics that could be collected would contain records of pipes that had been discarded for one reason or another, and would contain, at best, only estimates of the remaining life of the very important part of all the pipe laid that is still in service. The estimates of the remaining life of this pipe cannot be considered to have much value, and averages based on them are certainly no better. Even if the average life of pipe could be ascertained in some statistical way, it would be of little service. What is really wanted is the average percentage payment on the value of all the pipe to be made each year to make good the depreciation lost in it, and this percentage is undoubtedly quite different from the depreciation computed by the sinking-fund method from the average life of pipe, if it could be ascertained.

The writer believes that in the case of cast-iron pipe the gradual reduction in carrying capacity, the increase in leakage from the joints, and the fact that a system now designed might better serve its pur-

pose than an old system, are the dominating factors, and that the elapsed period of life is of secondary importance.

It may be suggested that in using the sinking-fund method it is possible to assume a life for pipe to give any rate of depreciation that is desired, and as the assumed life is pretty sure to be beyond the experience of the ordinary water-works system, it cannot easily be shown to be fallacious. Thus, with a 4% sinking fund, the depreciations corresponding to several lines are as follows:

30-year	life,	1.78%	annual	depreciation
40- "	"	1.05%	"	"
60- "	"	0.42%	"	"
80- "	"	0.18%	"	"
100- "	"	0.08%	"	"

Of course, the percentages given refer to the structures only when new. By the sinking-fund rule they would steadily increase with increasing age.

The ease of selecting a life to give any desired rate of depreciation is obvious. Considering the great difficulty, or perhaps rather the impossibility of securing statistics to show the true average life of cast-iron pipe, it may be questioned whether, as a matter of fact, the process has not been reversed, perhaps unconsciously, by estimators, and a life selected which would give a rate of depreciation which has seemed to them reasonable, as judged by other standards. If this is really the case, then the use of the sinking-fund method for computing depreciation may be regarded as a kind of mental exercise tending to familiarize the estimator with the workings of compound interest during long periods, and thereby to aid in keeping him from reaching unreasonable conclusions.

The sinking fund has this to recommend it: that it corresponds with practical experience to the extent that depreciation in the early life of a structure goes on slowly, and that as the years go on the rate is increased. There is a tendency to substitute a rule of calculation for judgment, and, as far as the basis of the rule can be well established, this is desirable; and, if a fixed rule is to be used, probably the sinking-fund method of computation is the safest that has been thus far proposed, but it must not be supposed that the real basis of the sinking-fund method is the actual determined life of the structures. It is, instead, the life which practical appraisers find will give rates of depreciation that are in accordance with their experience.

The writer believes that the most logical way to estimate depreciation in the appraisal of a plant is to consider each item on its merits, to determine and take into account its physical condition, its usefulness in the service and the state of the art, and the possibility of substituting other and more efficient structures to perform the ser-

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vice rendered by it. The excellence of the design must be especially considered, and depreciation may be several times greater on a clumsily designed structure than on a well-designed one, although the durabilities and lives may be the same. In a similar manner, a well-built structure suffers less depreciation than one not so well built.

Improvement in methods of pumping, filtering, laying pipes, or performing any part of the service, all have their bearing on the depreciation of similar parts of an old plant. The rule must always be to find what deduction would fairly be made because of all the conditions that exist between a willing seller and a willing buyer fully informed as to those conditions.

In estimating the amount of depreciation to be written off each year, the writer believes that one of the best kinds of information is to be obtained from the financial histories of old plants in connection with appraisals. The question may then be taken up as to whether an annual depreciation of 1% on the cost of reproduction of the property from year to year, made during the whole life of the plant, would have served to have brought the book value to an amount reasonably near the estimated cost of reproduction at the time of the appraisal, and if not, to find as nearly as may be what percentage thus allowed annually would have sufficed to do this. Data of this kind are hard to get, but the engineer having to do with appraisals occasionally has opportunities to secure them. Valuable as these data are, they must be used with caution, as the depreciation of the future is not necessarily measured by that of the past.

The method of allowing for depreciation in the annual operating account is an important matter. The writer recalls the case of trustees who had undertaken the management of an important water-supply system, and who were confronted on the one hand with the difficulty of raising money for urgently needed extensions, and on the other of being required by law to take up a business entirely foreign to that for which they were appointed, namely, that of setting aside a certain proportion of the income from the operation of the property and investing it in securities to be held against the redemption of bonds when they became due. The situation was a trying one, and one that ought not to have existed.

The sinking fund was invented to protect the bondholders who might think that otherwise the city would not be able to raise so large a sum of money on the maturity of the bonds. That is the only possible justification for its existence. If we grant the solvency of the city or company, and assume that they will be able to refund the bonds when they become due, it is better in every way that the allowance for depreciation, or the sum that would otherwise be put into a sinking fund, should be invested in the plant as far as there is need of it.

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The author does not know of any sinking fund that has been established for the especial purpose of meeting depreciation. Such a fund as he suggests would be one to which annual depreciation allowances would be paid, and from which money would be taken to replace worn out or discarded parts, and the amount of such a fund, he thinks, would not often exceed 10 or 15% of the value of the plant. The writer feels that the question of whether such a fund is necessary or desirable must be left to those who operate the plant. If it is desirable, he suggests that it might be more appropriately called "working capital", as its function would be quite different from that ordinarily covered by the term, "sinking fund".

It is the writer's feeling that, so far as such a fund of working capital is reasonably necessary and desirable for the proper and economical administration of the plant, the amount of such fund might properly be taken into account in fixing the rates that might be charged without exceeding those that would yield a fair return on the property; but he sees no reason for taking into account otherwise any part of the value of the original plant that may have been lost by depreciation in making such rates. It is clearly the privilege and duty of public service corporations to charge rates that will permit marking off a proper allowance for depreciation. When this allowance has been once determined and marked off, there would seem to be no equitable reason why the capital thus marked off should be further considered in connection with the question of fair rates.

Mr. Vensano.

H. C. VENSANO, ASSOC. M. AM. SOC. C. E. (by letter.)—in looking over this interesting paper, the writer was surprised to find the argument that, unless a sinking fund for depreciation is actually in bank or in trust, it should not be considered a reason for not deducting depreciation in valuations for rate-making purposes.

In his summary, the first and second arguments indicate that Mr. Alvord feels that even if a sinking fund exists, reinvested in the plant itself or elsewhere, it should not be considered as such.

To the writer it seems axiomatic that, if a sinking fund, properly handled and accounted for, exists anywhere at all, its existence is a good reason for the non-deduction of depreciation in valuations for rate-making purposes.

If a sinking fund to cover depreciation has been provided, and if it has been properly handled, and if, as a matter of course, the returns from such fund, in no matter what form they are received, have been turned back into that fund, it would seem clear that it is immaterial whether such fund exists in bank and trust, in the property itself, or elsewhere.

Mr. Alvord seems to feel that, even with proper funds reinvested in the property and accounts properly kept, such accounts are merely

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the opinion of the owner as to depreciation which has accrued on his property, and further argues that when a fund is handled in this manner the owner will have a return on it as a sinking fund and "at the same time have a return on it as a betterment to the plant," and that "this would be an obvious injustice to the public."

If the fund is properly kept, the accumulations from such investment, of course, will go into the sinking fund itself, and no portion of it will be returned as dividends to the owner, and in such case no injustice to the public would follow and no duplication of returns on investment would be made by the public to the owner. The whole point hinges on the proper accounting, and with such proper accounting the public's interest will be entirely safeguarded. As a matter of fact, any sinking fund, whether in bank or elsewhere, is now largely an "opinion of the owner" as to the depreciation on his property, or rather on the rate at which it is likely to occur, and will always represent either the owner's opinion or the opinion of some rate-making or controlling body which he has accepted, or has been forced to accept.

Assume a case in point, and see if any excess burden is thrown on the public. Take a large utility of complex properties, which utility may be considered of such a nature that after a number of years the necessary yearly depreciation replacements would arrive at practically a constant amount (assuming no extensions or betterments as occurring). Assume an original investment of $100 in complex properties having an average life (say, 40 years), such that $0.83 set aside annually as a depreciating increment will produce the proper sinking or replacement fund. Now assume that the property has been in existence so long that annual depreciation replacements have become constant (no betterment or additions considered), so that at this time the sinking fund in bank or elsewhere has reached a total of $33.40, all replacement made in the meantime having been paid from it, and that now the balance has been reached such that the annual increment set aside and the accruing interest on the $33.40 will just pay for the annual replacements, that is, the cost of annual replacements has practically become constant.

Also, for convenience, suppose that the interest rate on this sinking fund (assumed to be in the bank) is the same as the dividend rate to the owners of the utility. Now, in this case (Case 1), the fund is actually in bank, rates of dividend and interest are 5%, and rates have been fixed by a rate-making public service body so that they will provide such dividends plus the necessary depreciating increment of $0.83, as previously mentioned. Table 1 then shows the result.

Now (Case 2), it may also be that extensions and betterments have been made so rapidly that the sinking fund, instead of having been banked, may have been reinvested in the property itself; let it be supposed that this has been done. The public, of course, will furnish

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TABLE 1.—CASE 1.

			Paid by Public.		Paid by Bank.		Available for Replacements and Reserve.		Available for Dividends.	
			Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
Original investment	$100.00	Allowed as profits	5%	$5.00						$5.00
		" for depreciation reserve	0.83%	0.83				$0.83		
Sinking fund in bank	33.40	Interest			5%	$1.67		1.67		
Investment in betterments, through later security issues	33.40	Allowed as profits	5%	1.67						1.67
		" for depreciation reserve	0.83%	0.28				0.28		
Total paid by public				$7.78						
" " " bank						$1.67				
		Total for depreciation reserve						$2.78		
Total investment	$133.40	Total available for dividends							5%	$6.67

TABLE 2.—CASE 2.

			Paid by Public.		Paid by Bank.		Available for Replacements and Reserve.		Allowable for Dividends.	
			Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
Original investment	$100.00	Allowed as profits	5%	$5.00						$5.00
		Allowed as depreciation increment	0.83%	0.83				$0.83		
Depreciation reserve reinvested as betterments	33.40	Allowed as profits on depreciation reserve	5%	1.67				1.67		
		Allowed as depreciation increment	0.83%	0.28				0.28		
Total paid by public				$7.78						
Total investment	$100.00	Available for dividends							5%	$5.00
		Total to depreciation reserve						$2.78		

NOTE.—The total, $2.78, to depreciation reserve, represents $2.50, the constant increment on original investment (after 40 years life) and $0.28, a depreciation increment on depreciation reserve (Case 2), or investment in betterments (Case 1). This is proportional to depreciation increment originally provided on original investment.

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the returns through the rates. The results are shown in Table 2. In Table 1, it has also been assumed that similar betterments have been made through money provided by an issue of new securities.

For proper accounting, if, in Case 2, $33.40 reinvested is still maintained in the accounts as a depreciation fund and its increment of earning is returned to such fund (as would be the case were such fund held in bank, as in Case 1), and if the dividends are then paid as before on the actual investment, exclusive of the sinking fund, it will be seen that the public pays and the owners receive the same amounts in each case.

It must be borne in mind that the annual depreciations on additions or betterments have not arrived at a constant stage as yet, and therefore the same proportionate annual expenditure for replacement is not necessary on such part of the property as on the original assumed properties. It is to be noted, however, that there has been provided and set aside a depreciation increment on the amount spent in betterments, which is exactly proportional to that provided on the original investment. Therefore, if the proper increment, or proper average life for the original investment has been assumed (the additions and betterments, of course, being of the same class of properties), the proper sinking-fund increment has therefore also been provided on such additions and betterments.

This would seem to show clearly that, with correct accounting, sinking funds may be very properly reinvested in the property without extra cost to the public. As a matter of fact, as Mr. Alvord pointed out, the returns on a bank sinking fund will be less than the returns from similar funds reinvested in the property itself, and therefore the public would ordinarily actually be benefited by such reinvestment, in that the depreciation increment, which must be included in the rates paid, could be reduced.

This should show clearly that a reinvested sinking fund is certainly not a matter of mere bookkeeping.

It may be argued that the fund is not as safe when thus reinvested, but the assumption must be made that, under public service rate regulation, the rates will be adjusted upward when necessary as well as downward, so that a constant and safe return will be obtained from public utility investments, and, if this is done, the reinvestment of money in the property will be the safest possible method of handling such funds.

If public utility investments are not made safe under Government regulation, Government regulation will certainly fail as such, and either destroy the utilities or necessitate Government ownership; because, with uncertain interest returns, held at a low rate, no capital will be obtainable for investment in such utility.

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It may also be argued that, up to the time where dividend returns on the sinking fund have reached the state of balance and become sufficient to cover the annual depreciation replacement, a fund reinvested in the property is not in such shape as to afford ready use as cash.

It is to be noted, however, as Mr. Alvord says, that large replacements can generally be foreseen a number of years in advance, and a new issue of securities can then be provided to cover such replacement; the betterments constructed under the original sinking fund can be then transferred to capital account under the new securities issued, and the depreciation account credited with the replacement expenditures.

A similar reasoning will hold for any outside investment, provided such investment is safe, and provided the returns from such fund are turned back into it and the whole held as a part of the utility, and, as an additional advisable qualification, that such investment be in liquid form. That is, it might be an investment in bonds of other companies which have a ready sale, and may therefore be considered as liquid. Certainly, no one could object to an investment in Government bonds as being any less safe or less real as a sinking fund than such fund held in bank or in trust.

The writer would also like to add that, in addition to believing that any properly kept sinking fund is a good reason for not deducting depreciation in valuations for rate-making purposes, he also believes that, even in some cases where such fund does not exist, it may be proper to omit the deduction of depreciation. Perhaps it would be better to say that the writer believes that depreciation in general should not be deducted, except in cases where it is clearly shown that the property has been mishandled by the payment of excessive dividends, or mismanaged in some other way, whereby the accumulating of a sinking fund has been prevented, or when such fund has been mismanaged.

Take, for instance, the case of a company which has gone into business recently in proportion to the average life of its properties and equipment. Assume that such company has been in business 10 years, with an average property life of 30 years. In many cases the property might not have paid during its early life, and with the rates at present in force has just arrived at the point where it is able to set aside money for a replacement fund and to pay dividends. Assume that the earnings are sufficiently large to supply a depreciation increment which in the remaining 20 years of life will give the total fund required, which fund, theoretically, should have been started 10 years previously with a smaller depreciation increment. Would it be fair, then, to such company, which has received no return on its investment, and has been unable, because earnings were too small, to provide deprecia-

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tion increment, to cut its rates now to a basis fixed by a depreciated valuation. Such property without regulation could no doubt be considered a good and well-handled investment, as business investments go, and still would be placed among the class of failures if its rates were calculated on depreciated valuation, as just shown. In any event, if rates in this case are to be based on such depreciated valuation, it would seem only equitable to make them sufficiently high to afford a depreciation increment large enough to return the entire investment in the remaining life of the plant.

It is admitted, of course, that, in using valuations from which depreciation has been deducted, rates can still be fixed logically and equitably by a proper handling of such depreciated valuation and by allowing earning to cover a depreciation increment based on the remainder of the life of the property, as shown by Mr. Grunsky in the paper to which Mr. Alvord has referred.

The whole question as to whether or not depreciation should be deducted from physical valuations for rate-making purposes would seem to the writer largely a matter as to whether or not the public utility had been handled correctly. It is his opinion that Mr. Grunsky's theory is entirely correct, and, as theory, could be applied to any rationally handled property. It would seem that no theory of any kind can be made to apply to a property which has not been handled reasonably and honestly by its management, and at the same time be made to apply equally to a well-managed property.

Where a property has been earning large returns, and has disbursed them as fast as earned, and over and above a reasonable dividend rate; and where, at the same time, no sinking fund has been provided, it is clear that such company has shown itself ignorant of depreciation results as a whole, and therefore must suffer the penalty.

This, however, would not seem to disprove the theory as such, and as argued by Mr. Grunsky, that, in general, depreciation should not be deducted in physical valuations for rate-making purposes.

Mr. Metcalf.

LEONARD METCALF, M. AM. SOC. C. E. (by letter).—Mr. Alvord's interesting paper illustrates anew by what different mental paths different men have reached similar conclusions.

As the writer's views on depreciation are consonant with those set forth by the Special Committee of this Society on the Valuation of Public Utilities in its report which will be in print before this discussion, the writer does not purpose to do more than call attention to certain points in the application of the equal-annual-payment method of figuring depreciation, as compared with the ordinary sinking-fund method referred to by Mr. Alvord. The fundamental difference in the two methods consists in the fact that under the equal-annual-payment method, depreciation is treated as an annual amortization or repayment of capital (renewals *per contra* being treated as new con-

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struction), and the necessary accretion figured on the sinking-fund basis is assumed to be paid to the owner with the annual uniform depreciation allowance, figured on the sinking-fund basis; whereas under the ordinary sinking-fund method of treatment the necessary annual contribution is made to a sinking fund which must be kept intact until the amortization of the item of property under consideration, in order that the fund may earn its own accretion. Under the former method the depreciation allowance is actually deducted from the value of the property, and the fair return is predicated on its depreciated value. Under the latter method, although the fair return is predicated nominally on the depreciated value of the property, the sinking fund must be allowed to earn its accretions, and, therefore, in effect, the fair return, until the sinking fund on any item of property is complete, must be based on the full original value, as otherwise the owner will be denied a portion of his legitimate return. An examination of the table in the Committee's report showing the annual depreciation allowances on an item of property having a life of 20 years, will make this clear.

Under the equal-annual-payment method of treatment, the annual depreciation allowance is increased each year over that of the previous year by the amount of interest or accretion on the payment of the last year, increasing in an item of property having a 20-year life, for instance, progressively from $3.02 per annum on the first year to $7.64 on the last year, if figured on a 5% rate, the sum of the total payments amounting to 100% in the 20-year life. This is in effect equivalent to figuring the depreciation by the sinking-fund method on the original value of the property throughout the period of life of this item, as would be done by the sinking-fund method, it being assumed, however, under the latter, that the sinking fund itself shall earn the accretion.

Why, then, it may be asked, is the equal-annual-payment method to be preferred to the ordinary sinking-fund method? The answer is to be found in the fact that the accounting is perhaps somewhat simplified, and, most important of all, that the combined payments of depreciation and fair return are, with due consideration of the increasing cost of repairs during the life of the various elements of the property, made as nearly uniform as practicable, or, at all events, are made much more uniform than by the sinking-fund method as ordinarily applied.

In its practical application, it has seemed to the writer likely to prove advantageous to establish the approximate allowances to be made for depreciation at intervals of 5 years, more or less, and to convert the estimated allowances or rates of allowance into percentages of gross revenue, rather than percentages of reproduction cost, purely on grounds of expediency and convenience in application. Under such a method it might be found, for instance, that 10% more or less per

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annum of the gross revenue should be charged off to depreciation or amortization of property—renewals, as previously stated, being charged to new construction. Thus the depreciation allowance would grow annually with the increase in revenue, which is, of course, accompanied with increase in investment and value in property.

To attempt to apply the depreciation allowance as a percentage of reproduction cost involves keeping most careful track of the various elements of property or groups of property having lives of different estimated lengths, adding to and subtracting from these groups new plant built and old plant abandoned—a very laborious and difficult accounting problem. Under the other method of application, book accounts can be kept of the deduction on account of depreciation and memoranda of the property which has actually gone out of service. At the stated periods, then, the more difficult task of comparing the assumptions with the facts developed in the 5-year interval can be accomplished, and any necessary changes in rate allowed can be effected.

Under the equal-annual-payment method and commission control, it does not matter seriously if the allowance for depreciation is somewhat larger than necessary, provided it is not seriously burdensome in the rate, for the reason that the rate-payer immediately benefits in the reduced revenue required, resulting from the amortization of a portion of the property. The owner, on the other hand, does not suffer, as he is repaid an equivalent in cash of the property thus amortized.

Although, theoretically, if no new construction were added to the property, the depreciated value of the property would gradually decrease, as also the fair return, practically, no such condition is likely to arise, inasmuch as public service corporation properties are operated perpetually, and with the growth which is enjoyed by the majority of American cities the renewals and the new construction required will more than offset the depreciation allowance, so that the actual depreciated value of the property will practically be likely to increase rather than decrease, as will, therefore, the amount of the outstanding securities of the corporation. If, for short intervals of time, this should not be the case, then the repayment of capital through depreciation account could be carried in the bank or otherwise invested temporarily until the need does arise, or it can be used in actually reducing the mortgage debt by a retirement of a portion of the bonds, which bonds could be re-issued at a later date when the funds are required.

In the sinking-fund method of applying depreciation, the easiest practical method of accounting the sinking fund is probably the investment of the depreciation account in the bonds of the company, if the funds cannot be absorbed in new construction, and the keeping alive of these bonds, with the re-investment of the coupon interest on them in depreciation account.

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Mr. Alvord's comment, that if depreciation allowances are not used by the owners of public service corporations in renewals, new construction, or amortization of debt, but are used instead for outside investment, the owners must make recognition of such deduction so far as the public is concerned, is sound; and if these depreciation funds be re-invested in renewals or new construction, the writer is of the opinion that a fair rate of return should be allowed on them as allowed on the remainder of the property, and not simply bank interest rates, for the reason that in such investment in the property, hazard is assumed which must be compensated, and it is further to be borne in mind that there is always some loss in interest in the delay in prompt investment of the sinking-fund surpluses which may accumulate from time to time.

In the case of the equal-annual-payment method, it becomes a matter of indifference to the public how the depreciation funds are utilized by the owners of the property, inasmuch as the value of the property on which the fair rate of return is predicated is decreased annually by the amount of this depreciation allowance, and it seems fair and safe under operating conditions to figure the accretions of annual depreciation allowance to be paid to the owner on the basis of rates at which money could be obtained by the corporation with ample security.

Mr. Jackson.

WILLIAM B. JACKSON, M. AM. SOC. C. E. (by letter).—It is not possible for any one who is truly interested in this subject to read Mr. Alvord's thoughtful discussion without receiving valuable stimulus.

In the case of well-established public utility properties, it does not seem reasonable to consider that a sum of money may with propriety be laid aside in a trust fund of which the amount is determined by the amount which the reproduction cost of the physical property new exceeds the physical value depreciated, in accordance with the rules laid down by Mr. Alvord. There may be possible cases where special conditions, such as hazards of the business, might make necessary the laying aside of a sum of such amount, or even more, but the depreciation element of the property, as defined in this paper—and with which definition the writer heartily agrees—can never justify such a trust fund. As must be readily appreciated by any one who has given this matter careful study, only a relatively small part of such an amount is ever required on account of depreciation in usual public utility properties, and, such being the case, how can any one justly permit of a sum of money amounting to from 10 to 15% of the value of such a property to lie for all time in a trust fund. That is exactly what would be done if a public service company permitted a sum equal to the aforesaid difference to lie in a trust fund, and did not differentiate between the part that might be necessary for use in

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replacement and that which by the nature of things could never be thus used.

In the matter as to whether, in determining the fair value of a public utility property for rate-making purposes, the reproduction value new or the reproduction value depreciated is more important in arriving at a fair value on which the rate of return shall be made, consider for a moment a public utility property from the time of beginning business. At the beginning the company has a right to earn its current operating expenses, a reasonable sum to cover depreciation expenses, a reasonable sum for reserves, and a reasonable return on a fair valuation of its property. If the company is unable to do this during the early years of its existence, it certainly in equity has a right either to add the deficits to its investment on which it may earn a reasonable return, or to have them made up in some other way. When the time arrives when the company can begin to make money, there is no practical reason to require that it shall build up a larger reserve fund than the exigencies of the business require; therefore, it is not logical to assume that the owners have taken out a part of the value of the plant when they have not built up a depreciation fund to an amount which is entirely unnecessary for the most economical and reliable operation of the property. It is equally illogical to assume that any part of the earnings of the property should be considered as constituting such an unnecessarily large fund.

The writer believes that undue importance is placed on depreciation of plant *per se* without due consideration being given to the question whether the property has been kept in such condition as to give substantially as good and economical service as a new property and is fully safeguarded by its reserves from financial breakdown.

In this the writer does not wish to imply any lack of appreciation of the vital necessity for a vigorous carrying out of a safe depreciation programme, and his position in these matters is fully attested in his paper on this subject early in 1910 before the Western Society of Engineers, and in his discussion written in 1911 for the American Academy of Political and Social Science. There is undoubtedly no more unavoidable expense to be met by any public utility company than depreciation expense, that is, the expense for renewals of outused plant.

Mr. Lavis.

F. Lavis, M. Am. Soc. C. E.—It has always seemed to the speaker to be much easier to visualize depreciation and place a value on it than to estimate appreciation properly. He is inclined to agree with the author, that it can be demonstrated, at least from a purely theoretical point of view, that when depreciation is not offset by an actual sinking fund, its value should be deducted in order to obtain the present value of physical property. When, however, valuations of railroads are made for the purposes of rate-regulation or rate-making,

Mr. Lavis.

it seems as if there might be good reasons for not making the deduction.

The discussion which follows is confined to the effect of this theory on the valuation of the properties of railroads, as the speaker's experience has been principally with this section of the industries of the country coming under the general heading of public service or public utilities companies, and though it is probable that the general principles may hold good in any case, there may be some inherent differences in connection with the properties of some of the other sections, such as lighting, water supply, etc., with which he is not so familiar, which when applied to them might modify the conclusions.

It also seems best to confine any discussion of this subject to cases where the depreciation is normal, that is to say, in the case of railroads, where the plant, roadbed, equipment, etc., is maintained in such condition as to give the public reasonably full and efficient service, which with the present close inspection and regulation by the National and State authorities, should be true of most railroads. Abnormal depreciation, in any event, requires special treatment, as a reduction of rates, of course, would not tend to build up a property which is badly run down.

The theory that depreciation should not be deducted when valuations are made for the purpose of establishing rates has been advanced by C. E. Grunsky and W. J. Wilgus, Members, Am. Soc. C. E., in papers recently presented to the Society, and by H. P. Gillette, M. Am. Soc. C. E., in a brief recently submitted to the Public Service Commission of the State of Washington. Mr. Alvord also points out other differences of opinion in regard to this matter, and intimates quite truly that a proper understanding of it is not easy.

If a sinking fund is maintained to cover properly such depreciation as there may be, there can hardly be a difference of opinion that rates should be based on the full value; to this the author agrees. If, however, a sinking fund is not maintained, he claims that the value is not in the property, and, therefore, the public should not be asked to provide interest on such full value. He points out also that, in the United States, sinking funds are not usually maintained. In reference to this it may be noted that a sinking fund of 10% on the capitalized value of our railroads would amount to about $1 500 000 000—much more capital than we can afford to have lying idle.

In order to understand the argument of those who claim that depreciation should be deducted, in cases where a sinking fund is not maintained, take a concrete example: for instance, a new railroad, the value of which, for convenience, will be assumed to be $100, and a fair return on the investment as 7 per cent. Assume, also, that the rates provide for the establishment of a sinking fund.

Mr. Lavis.

At the end of 10 years the road is rendering full and adequate service, though parts of its structure and equipment show signs of wear. The roadbed, etc., in the meantime has consolidated and its value has appreciated. Assume, however, for the purpose of the argument, that after allowing for the appreciation, etc., the net value of the depreciation is 10 per cent. The actual value of the property would then be $90 and sinking fund would be $10.

Under these conditions it will be admitted that the rates should still be maintained to provide the 7% on the amount of the full original investment, inasmuch as the sinking fund produces nothing.

During the period under consideration the annual revenue and expense account of the railroad might be approximately as follows: (Round numbers are taken for the sake of clearness.)

Operating cost	$12
Sinking Fund	1
Interest	7
Total	$20

Now, suppose that during this period the dollar, instead of being added yearly to the sinking fund, is paid out in dividends, which have been 8% instead of 7%, so that at the end of the 10 years the $10, instead of forming the sinking fund, have gone back into the pockets of the owners, then, it is claimed, the public should not be asked to pay rates on the full $100. If the case in actual practice were as simple as this, there would be little need of further argument.

The practice in the United States, however, has not been to establish sinking funds, which produce nothing, but often to put the money back into the property for betterments.

In the case of a new railroad, such as the speaker is considering, as a temporary measure, some of the slopes in the cuts are not fully taken out or not trimmed up until after it is put in operation; the line is not completely ballasted; settlement of embankments has to be taken care of, and so on. Much of this work is done during this first period with the 1% which would have gone to form the sinking fund. Later, the surplus earnings go to provide for reduction of gradients, improvements of alignment, elimination of level crossings, etc.

In Europe and in most other countries of the world, except in North America, all such work is usually charged to additional capital expenditures, and it is largely for this reason that the capitalized value per mile of railroad in Europe is so much greater than in the United States, the capitalized value of English railroads being almost twice as great. There is some idea that railroads in Europe, and particularly in England, have cost more because they are so much better; this,

however, is not so, at least not nearly to the extent of the very much greater capitalization. Mr. Lavis.

In regard to betterments and improvements paid for out of earnings, the argument, of course, is that in a properly made valuation of an existing property, the appreciation due to money spent for betterments and improvements will be duly allowed for, and that when it is, depreciation due to actual wear and tear should be deducted, in order to get the true fair value of the property in use.

If appreciation of the thousand and one improvements in the property of any well-established railroad can be properly allowed for, well and good, but, to the speaker, this seems at least somewhat difficult, whereas there is never any difficulty in allowing sufficient depreciation, so that, after all, the question resolves itself into the ability to make a true valuation of the physical property.

There is no doubt that a fair and just valuation, equitable alike to the public and to the owners, can be made, but that it can be mathematically exact is practically impossible, and because there is so much more probability of over-estimating depreciation and under-estimating appreciation, one may very well hestitate before deducting the former from a valuation to be used for the purpose of fixing or regulating rates.

In most of the valuations which have been made public, the so-called present value has been estimated to be from about 10 to 15% less than the estimated cost of reproduction new. In some cases appreciation due to the consolidation of the roadbed, etc., has been allowed, but apparently the net result is to show a present lesser value, and, according to figures generally accepted so far, the normal depreciation of a well-managed and properly maintained railroad is assumed to be about 10 per cent.

It seems to the speaker that this amount might well be, and probably generally is, offset by what might be called "not easily apparent appreciation." Any railway man of experience can easily imagine work of a physical character done on a well-established road, 20, 30, or 50 years old, which is not easily apparent, the worth of which, however, still exists.

In an article* describing the construction of a railroad recently completed near Kansas City, the Chief Engineer makes the statement that "Owing to the newness of the road, the train schedules are not yet as fast as they will be later."

In another article,† describing some recent new work on an improvement of the Nashville, Chattanooga and St. Louis Railway, occurs the following:

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* *Engineering News*, October 9th, 1913.
† *Engineering News*, October 23d, 1913.

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"On the new roadbed a heavy bed of cinders is used for ballasting, as this is more easily surfaced during the inevitable settlement of the fills, and forms eventually a good sub-ballast between the roadbed and the permanent stone ballast."

These, of course, are only specific instances of what all experienced railroad engineers know, but it seems as if the importance of this phase of the valuation question is not always fully realized, largely because many of the values thus introduced into a railroad property are not easily apparent.

There is no possible doubt that any of our modern trunk-line railroads are better transportation machines as they stand to-day, with rails and ties partly worn, with other parts of the equipment showing some wear, than they would be if we were able to replace every part in a brand-new condition. Theoretically, by taking specific instances of parts of the railroad machine, such as rails, ties, or rolling stock, one can easily visualize and value the depreciation, but it is known that the well-established railroad, as a whole, is a better and more efficient machine to-day than it was the day it was built.

Knowing this, and there can be no doubt of it, there is something wrong somewhere with a theory which gives a lessened value to a machine which is better than new, and it seems to the speaker that the trouble principally is, not the fact that depreciation is calculated and allowed for, but that appreciation is not appreciated.

In estimating the value, whether for rate-making or for any other purpose, of the purely physical elements of the property of a railroad, depreciation may be estimated and deducted, but one should also be sure that full value is given to the appreciation, to the development costs, and to the other elements which go to make up the efficient transportation machine.

Mr. Humphreys.

ALEXANDER C. HUMPHREYS, M. AM. SOC. C. E.—The reading of this paper should help to emphasize a responsibility which to-day rests heavily on the engineers of the United States; namely, to educate the public and their representatives in legislatures, commissions, and Courts, many of them necessarily ignorant on the questions involved in valuations and rate-making. Here we, the engineers of the country, are charged with an especial responsibility because of our specialized training. First we should, among ourselves, do our utmost to form right judgments on these momentous questions. It is with great regret, therefore, that the speaker finds himself almost completely out of accord with the views expressed in this paper.

We are to-day passing through a period of trial for our form of government. Therefore the speaker feels particularly that, with regard to all public questions, we must frankly express our honest opinions.

Mr. Humphreys.

This is the only way left to meet effectively the mass of bad advice which is being fed to the public.

In the first lines of the paper it is stated: "that space will not be taken here to review fundamentals". The author then proceeds to discuss the very fundamentals of the subject of depreciation, and to support views which, in the speaker's opinion, are fundamentally wrong.

As the speaker understands the paper, the author is attempting to confine himself practically to "the relation of depreciation to cost new", which, he says, is "vital", and that only one line of evidence (reproduction cost new) is to be considered. The speaker is not prepared to agree that "other lines of evidence are equally important before arriving at value".

Mr. Alvord then refers to some writers who have and have not agreed with him, and continues:

"Here, therefore, there seems to be a very serious disagreement, among authors who have written recently on this subject, as to whether or not depreciation should be deducted from the cost new of a property for finding value for rate-making purposes where the reproduction method is adopted as a basis of value. It seems to be desirable, therefore, to present a further study of this question."

Certainly, this paper makes it more than ever desirable that there should be further study of this question. Although the speaker is overwhelmed at the present time with duties accepted, he feels that he would be delinquent if he did not contribute his little share in this much needed discussion.

Mr. Alvord says:

"The sinking-fund method seems to be an accurate way to compute depreciation where all or most of the life history is known, as it consists largely in finding past depreciation."

First, the speaker objects to the use of the term "accurate" in connection with estimates on depreciation to accrue. Again, he objects to the intimation that, it can be assumed as a working basis that "all or most of the life history is known." This life history in large part must depend on the books of account and other records, which frequently are incomplete, and, even when complete, have been influenced by many changes in the system of accounting.

Again, in many cases, the existing company is the consolidation of many smaller companies, the books of which are often not available, and if available are found to be not self-explanatory. It would appear as if it should not be necessary to say to men of experience anything more, or even as much as the speaker has said, on this point.

The author then says:

"The sinking-fund method of computing or allowing for depreciation consists in determining the proper useful life of the structure or machine under consideration".

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Here he appears to be confusing estimated accruing and accrued depreciation with actual depreciation; but let that be as it may, who shall say authoritatively what is the "proper useful life" of any part of a plant. There are those who assume that the life of parts of plant can be found by reference to tables prepared by so-called experts; but surely this absurdity cannot be supported by men who have had experience as constructors and operators. To determine how much should be allowed for annual expenses or cost, we may well estimate the life of each part of a plant, knowing that these figures may prove to be far from the fact; and hence we are justified in making a liberal estimate for this purpose.

Here, however, we must never forget the underlying reason for this estimate and the journal entries based thereon, namely, to spread the cost of final renewals as uniformly as possible over the periods benefited, so that we shall not deceive ourselves as to losses and consequently as to profits. Especially should we be careful not to over-estimate our profits, and hence we should be sure not to under-estimate the cost of final renewals.

The speaker wishes that the term "depreciation" had never found favor. Much of the present confusion of thought on this subject would be avoided if only the term "final renewals" were used.

Here at once it is seen that such an estimate should not form the basis for determining present depreciation. This should be determined as closely as possible by the exercise of engineering, managerial, and accountancy ability, based on academic and practical considerations, chiefly the latter.

Here we have the all-important difference between estimated accruing depreciation as an element of cost, and actual depreciation in establishing the present value of an asset; this last by actual expert examination, having in mind physical decay, obsolescence, and inadequacy.

Again, Mr. Alvord says:

"In case a portion of the life has passed away, the present value of such structure is assumed to be, in a general way, the present cost of replacement new, less the accrued amount of the depreciation fund to date".

The speaker has just shown why the amount in the sinking fund may properly be quite different from the actual depreciation. It is based on an estimate made in advance, and on many assumptions. In making an appraisal we have before us the present facts as to physical condition, obsolescence, and inadequacy. In the first case we have foresight, and in the latter we have the advantage of backsight.

To determine the actual cost, would one go to an engineer's estimate of the cost of some large construction, or to the record of the money actually expended, which would cover all omissions, contin-

gencies, overhead charges, etc.? If the first course were taken, the speaker fears that the business world would be more than ever critical of engineers' estimates, which have so often misled investors because of the omissions referred to. Mr. Humphreys.

Supposing the existence of an approximately correct determination of the actual depreciation to date, the speaker contends most emphatically that, although perhaps the plant is thus reduced in value, no deduction therefore should be made in a rate-making case.

In meeting this proposition, the question is asked frequently: "Would you pay as much for an old plant as for a new plant?" and the questioner thinks that the question settles the matter.

Not so. As a purchaser, if there were any real depreciation, the speaker would claim a reduction. Why? Because he has to assume the liability for the final renewal of the plant. In a rate-making case, however, that liability still rests on the owner, and therefore the plant must be considered as maintained or to be maintained by the owner.

Mr. Alvord, strangely enough, admits that "no deductions from cost new or reproduction cost, where that method [actual establishment of a sinking fund under trust conditions] is being used, would be necessary". Then he says:

"Where a proper depreciation fund is not set aside actually, but is only estimated and written off in the accounts, there is no such condition, and the matter of writing it off by the owner in his books amounts to only an opinion of his as to what such depreciation fund should be, but does not at all produce such a fund, or make it available; in consequence of which, an appraiser, using the cost new or reproduction method as one of the means of arriving at the value of the property, and finding no depreciation fund actually in hand or in trust, is obliged to deduct it from the property."

Is the appraiser "obliged" to deduct because of an opinion thus expressed?

Mr. Alvord shows that the owner is alive to the fact that depreciation is to be cared for, and that he refuses to inflate his profits by ignoring that fact. By his act he shows that he is liable for the cost of renewal, and that this item of cost must be obtained from income. A buyer would trade on this admission to get a lower price, but, as before shown, even then it by no means follows that the seller would have to deduct the full amount thus written off. That is not a question of the fair determination of values and the fair fixing of rates, but of bargaining.

Mr. Alvord then proceeds:

"In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand. A clear perception of this principle will save much confusion of thought."

Mr. Humphreys.

Here Mr. Alvord mistakes his assumption for an established fact. The speaker fears that that is a weakness throughout the paper.

The Standard Dictionary defines "Principle", in part, as follows:

"1. A source or cause from which a thing proceeds; a power that acts continuously or uniformly; a permanent or fundamental cause that naturally or necessarily produces certain results. * * * 2. That which is inherent in anything, determining its nature; essential character; essence."

Here, then, it is found that, notwithstanding his disclaimer, as before referred to, Mr. Alvord is discussing fundamentals.

The speaker presumes that much confusion of thought might be saved if all were prepared to adopt blindly Mr. Alvord's dicta and then refuse to confuse their minds by further independent thinking.

Now, to come down to facts: Assets are widely variant in character. Cash is an excellent asset, if the money is honest money. Particularly is it valuable in emergencies because it is liquid. Usually, it is a promise to pay, on the part of the government, of a bank, or, if not in actual cash, it might be a promise to pay by the trustee of a fund; but government or bank promises to pay are not always made good. There has been no little repudiation, even in this enlightened country. Or, government notes may be greatly depreciated. We have also had that experience in the United States, a condition corrected only by the courageous act of a great President now passed to his reward. Still more, the trustee of a fund may default—not an unknown occurrence. Now, if the owner is solvent and agrees to make good the loss due to depreciation, that is, to renew the plant, and this assumed liability is shown on the books, there must be a corresponding asset, or else money not earned has been paid out in dividends. The entries referred to, however, have been made to guard against such a ruinous course. Naturally, the speaker is not referring to fraudulent bookkeeping or fraudulent management, but is following Mr. Alvord in assuming normal conditions and management.

This asset, if not in cash retained, will in all probability appear in additional plant paid for from earnings, and may be regarded as borrowed from the depreciation reserve. The fact is that where the depreciation reserve is treated in this way it has to be borne in mind that all parts of plant which appear in the appraisal inventory as having been paid for from the depreciation reserve fall in one of two classes:

1.—Parts which have been installed as renewals or replacements, and are, therefore, in place of parts represented in the original investment, and hence to be included as such;

2.—Parts of plant which have been installed as extensions or betterments, and have been paid for with money borrowed from the depreciation reserve.

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These parts, therefore, should be included in the inventory and appraisal, because they are not duplications of investment, but represent additional investment.

In the case of an interest-bearing depreciation reserve, the balance to the reserve should be credited each year with interest at the rate agreed on. Then the money thus borrowed has to be returned when required for final renewals, and must be repaid from capital. In the meantime the company has been able to defer the day for final financing. These extensions and betterments can be considered as capital investments, the amounts borrowed from depreciation reserve standing as a liability against the owners. In reference to this matter, the point may be made again that, whether or not there is a depreciation reserve, the liability for renewals rests against the owners. The setting up of the reserve is, as before shown, to spread this item of loss as uniformly as possible over the periods benefited. The loss due to final renewals will occur in any case, though not exactly in the amount estimated.

Mr. Alvord then attempts to show that the deduction for depreciation would not lead to a progressive reduction in the rates. This discussion he carries on under the headings, "The Attempt to Simplify the Question", "Perpetual Life", "Utilities Commonly Require Growing Plants", "Earning Power of the Depreciated Plants", "Natural Accretions", and "Sinking Funds an Inherent Part of the Plant".

The speaker agrees with some of the statements he makes under these headings; but disagrees absolutely with his conclusions. On page 2053* the author states:

"It is clear, therefore, that, as far as valuation for the purpose of sale is concerned, it is right and proper that, in the absence of actual sinking funds or any sufficient reserve fund, they should be deducted from reproduction cost or 'cost new', when that method is used to ascertain value. Does not this also hold good in the case of valuation for rate-making?"

It is presumed that the author looks for a reply in the affirmative to this question. The speaker unhesitatingly replies in the negative; and this, it is thought, is where Mr. Alvord and his many adherents go astray.

The case of purchase and sale has nothing to do with the question, and the author has supplied, presumably unconsciously, many arguments to support this position. What he has to say under "Perpetual Life," can serve as an example.

What the public is concerned in is the quality of the service rendered. That is the test. It is for the commissions to see that the service is adequate in quantity and quality and that the investment and its return are not exorbitant.

* *Proceedings*, Am. Soc. C. E., for November, 1913.

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In purchase and sale, it is a case of bargain. It may be a case of turning a fixed asset into a liquid asset. There may be many reasons for the owner's willingness to sell well below value; but in any case he should be willing to deduct fairly for accrued actual depreciation, because the buyer has to assume the liability therefor. If the seller can turn over a sinking fund, which in the opinion of the buyer is a sufficient offset to depreciation, then his liability therefor is covered. All this provided this is the best bargain he can make; for bargain it is.

Under the heading, "The Practical Treatment of Depreciation Accounts", Mr. Alvord takes, as an example of the point he is trying to demonstrate, the case of "a railway having extra rolling stock which is only needed at certain seasons of the year when the traffic amounts to a maximum". Then he shows that the property would not be as valuable if this extra rolling stock were sold, relying on the chance of buying it back when actually needed. Again, the test is one of service. If this rolling stock had been sold before the appraisal were made, and not replaced, naturally the decrease in value would be reflected in the appraisal. If all the property had been sold, it would have a still greater effect on the appraisal, because no appraisal would be necessary.

To wind up this part of the discussion Mr. Alvord says:

"To state it in another way: a utility company plant is not as valuable to the public without a reserve fund for replacement as it is with such a fund intact and on hand and promptly available".

This is not stating it in another way, but is an entirely different proposition. The reserve fund is a matter of finance. The cars might not be purchasable for immediate delivery, but this by no means shows that money for replacements could not be obtained simply because there was not cash enough on hand. Such a case is almost unbelievable, provided the owner enjoyed the ordinary good credit of public service corporations. Again, it is a question of service, and the public is not interested in the financing of the replacements provided the amounts thus expended do not add to the capital burden.

Under the head of "Outside Investment of Depreciation Funds", Mr. Alvord objects to investing the depreciation reserve in betterments which might give 7 or 8%, instead of the guaranteed 3½ to 4 per cent. He refers to "the general risk incident to the enterprise". As all the capital invested is subject to this risk, there is no good in splitting hairs about this particular part of the investment; but Mr. Alvord thinks that it works an injustice to the public if the owner gets a return on these betterments. If the return has been allowed for in the amount annually to be set aside, it does not. The returns are in lieu of the interest otherwise allowed by the banker who holds the depreciation fund, and these returns are required to bring the reserve up to the required amount. As to the extra rate of earning, there

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can be no injury to the public if the property prospers because of this or any other saving. Also, the speaker imagines that Mr. Alvord has failed to appreciate the fact that, as a rule, extensions thus made do not pay a high return at the start and are frequently made in advance of a paying demand for the service.

The speaker, as a manager, has had to face this problem in not less than fifty cities in the United States.

Under the head of "Present Practice" Mr. Alvord says:

"The rate commissions have not yet discussed this question [that of deducting a 'theoretical depreciation fund'], nor has it been effectively presented to them, but it is believed that, when discussed, it will be along lines which are here indicated."

Believed by whom? The speaker quite disagrees as to the correctness of the statement. He can hardly think of any question which has been more discussed in the cases of which he knows. He also ventures to believe that the questions involved have been "effectively presented". The statements have not always been understood, for those to whom the statements and arguments were addressed were in some cases, through lack of training and lack of experience in the several lines of construction and administration, incapable of understanding the matter presented.

Under the head of "Should a Sinking Fund Earn Full Return", Mr. Alvord says: "We will have to await with patience the conclusions of the older commissions which are giving this matter study". Again, the speaker completely disagrees with him. It is not for us to "await with patience", or with impatience, the conclusions of the older commissions, or the younger commissions, on any point. It is for us to go ahead with open minds to determine what we believe is fair and honest to the public and public utilities, and to do our best to educate the public and the commissions. If they are right and we are wrong, it is for us to acknowledge the fact; it is for them to do the same. It by no means follows that we are to accept as correct the rulings of the commissions simply because we have to govern ourselves thereby until these decisions are reversed. Even the Supreme Court of the United States is not infallible.

The speaker also objects to the reference to the "older commissions", as if they of necessity were most likely to give just decisions. That does not follow. There may be an old commission with a new personnel, and "the last state may be worse than the first".

Under the head of "Depreciation Accounts", Mr. Alvord indicates that in his opinion the cost of depreciation should be estimated liberally. He says, "Some allowance must be made for the unknown". Also, "From time to time adjustments will have to be made to correct for errors in judgment as to future life". This is quite true; but the unknown has not happened when the rate-making appraisal is

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made, and if no "adjustments * * * to correct for errors in judgment" have been made, how about deducting from cost new the full amount thus accumulated in the reserve?

In the speaker's experience the so-called experts testifying for the people deduct the depreciation according to an assumed "average life", and they do not worry about the "unknown" or "errors in judgment". They are so used to dealing both in the unknown and errors in judgment that they have long since ceased to worry.

On page 2060* Mr. Alvord says:

"When renewals are made, the depreciation account should be debited with the outside capital replaced in the plant by the owner, and credited by the cost of the improvement."

Is this one transaction which is referred to? Is it a replacement? If so, is it an improvement? Of course, a replacement may include in part the element of improvement, but this hardly seems to be what is intended. To what account is "the cost of the improvement" to be credited?

The speaker does not find this a self-explanatory direction for a journal entry, and would suggest that it be rewritten.

Now, as to the several items of the summary:

1.—This is incomplete.

2.—The speaker disagrees with this, absolutely.

3.—"All the facts of the past are" not known. The sinking fund is not the most accurate method of computing depreciation. The depreciation has to be estimated before the sinking fund can be computed.

4.—This is quite correct, and furnishes an answer and contradiction to much that is in the paper.

5.—The reference to "private gain" is not clear. If the speaker understands the statement, he considers it an unfair proposition.

In his concluding words, the author says: "all the principles which it is endeavored herein to reason out are not yet generally accepted".

The speaker is glad to be able to agree with Mr. Alvord here; but must go farther and express the hope that most of these "principles" never will be "generally accepted". They cannot be accepted without working confiscation in some degree.

In conclusion the speaker begs to refer to a paper recently written by him which discusses at length most of the questions raised by Mr. Alvord. This paper is entitled "Depreciation, Estimated and Actual." It was presented in June, 1913, before the Institution of Gas Engineers of Great Britain, and has been reprinted in many of the technical journals of the United States, and also in pamphlet form.

* *Proceedings*, Am. Soc. C. E., for November, 1913.

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In this paper the speaker supports the proposition that no deduction should be made for depreciation if the property has been adequately maintained, and particularly that no deduction should be made on account of aging of the plant. He draws the line sharply between estimated accruing depreciation and actual present depreciation, the latter as found by expert examination at the time. He also shows that in not a few cases there is no need for any estimated depreciation because the current final renewals make a fairly uniform annual charge, and this applies to properties as a whole, and in many more cases to parts of properties, so that those parts can be eliminated from the estimate of accruing depreciation.

Mr. Floy.

HENRY FLOY, ESQ.—Without discussing in detail the points made by Mr. Alvord, some of which are well taken, the paper, as a whole, is wrong in its conclusion and unfair both to the utilities and the public.

The author bases his paper on two assumptions—which he does not attempt to prove—both of which are false:

First.—That both newly originated and long established utilities must be treated alike.

Second.—That a depreciation reserve fund is a necessity.

Before undertaking an intelligent discussion of this subject, there must be drawn a sharp line of demarcation as to whether the valuation being considered is that relating to an existing and long established corporation, or one which has originated under public service regulation. The same rules may not fairly be applied equally to utility corporations so differently originated.

In a case of property constructed and put into operation before the creation of regulating commissions, the investors, with the consent of municipal, State, or Federal authorities and the sanction of the public at large, were allowed to do certain things which are no longer permitted under utility regulation. Where a corporation is created and the money of the owners is invested, after notice, under conditions prescribed in advance by regulating bodies, there can be no fair or honest objection to the insistence on accumulation of depreciation funds out of revenue and the use of such funds in any reasonable way, but they cannot necessarily be used as a measure of the value of the property on which rates are to be fixed. With respect to corporations that have been in existence for a considerable period of time without commission regulation, and have been following certain generally accepted principles with the consent, more or less definite, of public authorities, the speaker takes emphatic exception to the proposition advanced by the author, that value for rate-fixing purposes must be determined by deducting the amount of a computed but not accumulated reserve fund from the cost of reproduction new. The arguments presented in the paper may be reasonable when applied to some

Mr. Floy. newly organized corporations, but they are unfair as applied to most existing corporations. The point to be made clear is that no *ex-post-facto* rulings should be attempted by commissions or countenanced by such organizations as the American Society of Civil Engineers, simply because no reserve fund is in hand. Who shall say that, because utilities had not accumulated reserve funds before public regulation was instituted, therefore, the corporations shall now be deprived of a part of their property. In determining rates where the public and corporations have consented to the existence of a certain thing in the past that is not legally or criminally wrong, and about which there still exists honest differences of opinion, it is not fair or right, in making rulings for the future, to attempt to apply such rulings to the past at the sacrifice of investment or the jeopardy of income. It is on this point—the difference between a corporation already in existence and one originated under new laws—that the mistakes have been made in the rulings of commissioners who, for the most part, inexperienced in utility corporation affairs, have attempted to apply theoretically perfected methods which are largely the result of development and experience. It is the old truth, that the same medicine will make some men well and others ill.

That the so-called, though misnamed, present value is not determined by the cost new less the amount of depreciation or reserve fund is indicated by the fact that a fund accumulating for a period of years, based on the original investment, would probably result in too small deduction from the reproduction cost new in case of investments which have enormously increased in value, and too great deduction in the case of those which have largely decreased in value.

In the second instance it may certainly be questioned whether there is any necessity for the accumulation of a reserve fund for the purpose of ultimately returning to the investor full value of his property, except in a case of limited franchises or where the lives of utilities are definitely determined. Although in most cases a moderate reserve fund should be kept in hand for contingencies, it is a fact, as has been argued for some years by the speaker, that there are utilities which do not require the accumulation of any depreciation reserve funds whatsoever. A corporation which has its property widely distributed, of various characters, with single units of relatively small value, may not require any depreciation reserve because the deterioration of property—after the first few years of operation—is taking place at an approximately uniform rate per annum, and, therefore, all depreciation becomes wear and tear and is taken care of as a part of the annual operating expense; as an example the Third Avenue Street Railway Company, of New York City, may be cited. This property consists of nearly 300 miles of surface track, partly overhead and partly underground trolley, with rolling stock to correspond of all

sizes and types, as well as storage battery cars, underground conduits, cables, overhead transmission systems, car barns, sub-stations, generating station, real estate, and other property extending from the lower end of the Island of Manhattan up to and through the Bronx and the County of Westchester, so that the exhaustion of any physical property, as far as can be anticipated, will not result in unduly increasing operating expenses for its replacement, or affecting net income, or in any way jeopardizing service or charges to the public. Despite the orders of the Public Service Commission to do so, the Third Avenue Company accumulates no depreciation fund, and yet is operating most successfully. Mr. Floy.

Why, then, should Mr. Alvord argue that this company should have on hand a reserve fund equal to 15 or 20% net value of its property, say, from $9 000 000 to $15 000 000, for which the corporation has no use, except to be allowed to earn fair return on this property under his theory.

On page 2055,* Mr. Alvord says, "a public utility owner, under such circumstances, is tempted to let his plant run down, and withdraw from the property the funds necessary for its renewal from time to time, and use such funds for his other gainful purposes." Except under abnormal conditions, this statement is not in accordance with the facts. Valuations of a large number of utility properties of different kinds, made by a host of independent and separate appraisers, result in showing that the average condition of these properties is between 80 and 90% of reproduction value after deducting the amount of depreciation, as determined by methods suggested by Mr. Alvord, or the straight-line method of calculating depreciation which shows even a smaller present value than Mr. Alvord's method. Consequently, it may be stated, without fear of contradiction, that the condition of the average utility plant is not run down, whether or not reserve funds are maintained. Moreover, utility regulation provides for proper maintenance and service, and this can be enforced and secured even to the exclusion of dividends. Take the case of a corporation which has issued only stock, and has no bonds or mortgage indebtedness outstanding—as is not unusual, particularly in New England—in such a case, the entire interest of the stockholders is a guaranty of good service. Why, then, must the public be further protected by an unwieldy and useless reserve fund, which is of no service to the corporation, except to permit it to earn on the full value of the property?

CLINTON S. BURNS, M. AM. SOC. C. E. (by letter).—The question as to whether or not depreciation should be deducted from the cost new of a property in finding the value for rate-making purposes where the reproduction method is adopted as a basis of value, is one that has Mr. Burns.

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often suggested itself to the writer, but not until recently has he undertaken to make a comprehensive analysis of the subject with regard to the equity of the points involved. Considering the case of the most simple proposition, as cited by the author, where the composite life of an entire plant has been estimated, at the end of which the plant would be subject to entire renewal; that is to say, the original investment will have completely disappeared at the end of this estimated life, the following analysis suggests itself:

First, let it be understood that the obligation to renew the property, or any portion thereof, is a liability of the public utility company, and if the company has not the cash in hand with which to meet this liability, it must assume the burden of financing the necessary renewals whenever the necessity may arise, and as far as the public is concerned, it is immaterial whether it chooses to provide in advance for the financing of renewals by setting aside its annual accretions to the depreciation fund, or, whether it chooses to make other uses of these funds and then re-finance the renewals when the necessity arises—provided, however, that it is strong enough to accomplish such re-financing promptly when called on to do so. Furthermore, it is now conceded by all authorities that the obligation to renew the property, or, in other words, to make good the depreciation, is very properly transferred to the public (the rate-payers). This is expressed in substance by the Supreme Court of the United States in the Knoxville case in the following words:

"The Company is not bound to see its property gradually waste without making provision out of the earnings for its replacement. It is entitled to see that from the earnings the value of the property involved is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning."

This language clearly defines the underlying principles which should govern an analysis of depreciation. Depreciation is an obligation to be met by the rate-payers—the public—at the time when renewals become necessary. Stating the problem in its simplest form, freed from all legal entanglements, and considering only the equity in the case, the following analysis results:

An investor, on entering the public utility business, dedicates his property to the public use, and, in return, the public obligates itself to pay a reasonable rate of return for the use of the property and to restore to the owner the value of any portion of it consumed or wasted in the public service; or, in other words, the public is obligated to make good the depreciation. Suppose, now, that the utility, at the beginning of the enterprise, should accept from the public the promissory notes of all the rate-payers, payable on demand, without interest, the face value of these notes aggregating an amount equal to the reproduction cost of the property, it being stipulated that the payments

would be demanded only as required for the renewal of totally depreciated property. If such a procedure were practical, and assuming that these notes were convertible into gold dollars on demand, at full face value, depreciation would thus be provided for; and an appraiser, called on to determine the value of the property for rate purposes, would undoubtedly recognize the injustice of deducting depreciation from the cost of reproduction in any project financed in this manner. Now, the foregoing procedure is exactly what the sinking-fund theory is, except that the investor allows these promissory notes to be paid in small annual payments, just sufficient so that if invested at compound interest, the total sum accumulated at the maturity of the notes (the expectancy of the property life), will equal their face value. The investor, therefore, in accepting depreciation on the sinking-fund basis, does not in fact, so far as its use value is concerned, have any of his available capital returned to him until the end of the life of the structure, and in order that he may receive a fair return on the value of his property, the rates must be based on the reproduction cost of the property, undiminished by depreciation. If the investor chooses to reinvest the cash on hand in the depreciation fund at any time, with the hope of securing greater returns therefor, he must assume the risk and incur the liability of returning the amount withdrawn plus its accumulations at compound interest when necessary for renewal expenditures; for example, if he is collecting depreciation on the sinking-fund basis, where 4% is considered a fair rate of interest at which to compound the fund, and he is able to withdraw the fund and invest it in extensions to the property, or elsewhere, and thus realize 8% thereon, the additional 4% is due him as compensation for the effort, risk, and worry inherent to the investment of capital, and the first 4%, being the proper rate for liquid capital, must be considered as going to restore depreciation. Mr. Burns.

The writer fails to see why any distinction should be made between the case where the sinking fund is actually kept in the bank or in trust and where it is kept merely as a book entry and has been diverted to other investments. In fact, the public, by the fundamental principles on which the sinking-fund theory is based, compels the utility owner to invest the depreciation fund somewhere, where it will draw 4% compound interest, or some predetermined rate considered proper for liquid capital; and how can it matter to the public whether it is invested in time deposits in a bank, in Government bonds drawing the necessary rate of interest, or in private venture where the owner perhaps hopes to realize 10%—and what matters it whether or not his expectations are realized? The public secures its recompense in being permitted to discharge its depreciation obligation by paying into the sinking fund the smaller annual contributions and imposing on the owner the obligation of reinvesting the fund; and the owner secures

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his reward for reinvestment if this venture proves successful, and he pays the penalty if it is a losing one.

The author states further "that the owner cannot have a return on it [the sinking fund] as a 'sinking fund' for depreciation, * * *, and at the same time have a return on it as a betterment to the plant. This would be an obvious injustice to the public." At first thought this seems plausible, but, on further consideration, is it not a fact that the contrary is the case, namely, that the public in reality owes this opportunity for reinvestment to the utility owner?

The public, having imposed on the public utility owner the necessity for reinvestment of the fund where it will earn, say, 4% interest, should in equity open the avenue of investment to him when the opportunity presents itself. To illustrate further, take the simplest case imaginable; for instance, a gravity system of water-works, where the entire property consists of $100 000 worth of wood pipe, having a life of 20 years; the utility owner dedicates this $100 000 to the public service, and, in return, the rate-payers obligate themselves to pay a reasonable return on the investment, and to preserve the property intact; or, in other words, make good the waste from usage. The rate-payers provide for the depreciation by giving their promissory notes aggregating $100 000 in face value, payable 20 years after date, without interest. The utility owner permits these notes to be paid in annual installments on a 4% sinking-fund basis, payable at the beginning of each year, or at the rate of $3 229 per annum; and it so happens that the betterments, which in this instance are for extensions to the pipe system, require the expenditure of $3 229 per year. The rate-payers, in consideration for being allowed to care for depreciation by paying the sum of $3 229 annually, permit, and, in fact, compel, the utility owner to reinvest this sum of $3 229 annually where it will earn 4% compound interest, and they also require him to invest this or some other $3 229 annually in pipe lines for extending the service to the new territory; but they pay him extra in consideration for these new extensions an amount equal to 3.229% to restore depreciation and, in addition thereto, a reasonable return on these further sums invested annually.

Two courses are open to the investor: he may leave the first $3 229 annually in a trust fund, earning 4%, in some reliable bank, and use $3 229 annually of his other resources in extensions, or, he may expend the $3 229 depreciation fund directly for extensions. In the former case he will find his situation in 20 years to be as follows:

He will have $100 000 in the trust fund with which to replace the original pipe system, and he is also in possession of $100 000 worth of extensions paid for from his outside resources. In the latter case, he will find at the end of 20 years that he has $100 000 worth of pipe requiring replacement, with no funds in trust, but he has the $100 000

in his outside resources which he may as well now spend all at once instead of at the rate of $3 229 annually, as was done in the former case. He also has the same investment in extensions or betterments as in the former case. Hence, the owner's financial situation is exactly the same at the end of 20 years regardless of which method of finance he chooses to pursue. The public is likewise in identically the same position in either case; for, had the owner left the depreciation account in trust until the $100 000 had accumulated, at the end of 20 years that sum is due the owner in exchange for his worn-out property, dedicated and consumed in the public service. Hence, the public could have no claim on the sum, nor title to any portion of it. Neither the owner of the utility nor the public being in any wise affected by the owner's withdrawal of the sinking fund for reinvestment or for extensions of the property, it would seem proper that the valuation for rate purposes should be the same in either case, no matter whether the sinking fund is left in a trust fund or used for private venture. If the amount of depreciation to be set aside annually is computed on the sinking-fund theory, it, therefore, seems that no deduction can equitably be made from reproduction cost in arriving at the proper valuation to use as the basis of rates. The rate-payers, therefore, should contribute annually $3 229 for depreciation and the reasonable rate for interest and profits to the owner, regardless of whether or not the depreciation fund is held in reserve. Mr. Burns.

The writer, therefore, can see no reason in equity for the conclusion that any injustice to the public results from allowing the owner of the utility to have return on the sinking fund for depreciation at the proper rate of interest for such depreciation accounts, and at the same time have return on it as betterment to the plant, for each betterment to the plant simply means a larger investment or larger plant on which the owner is entitled to the proper reasonable rate of return. Doubtless what the author had in mind was that it would be unjust to allow the owner to reinvest the sinking fund in betterments and receive thereon, say, 8% net return, and, at the same time, receive 4% as sinking-fund accumulation. It seems to the writer, however, that when the owner invests his sinking fund in betterments, or in private enterprise, and thereby receives 8% return, he in fact, does not receive 8%, but, on the contrary, receives only 4%, because half of this 8% return does not represent profit but simply goes to restore to him some of his property already consumed by the public in the public service.

If a community constructs a utility under municipal ownership, it is almost universally the custom to pay for such a utility by a bond issue. Now, to determine the basis of rates in such a municipal enterprise, in order that it shall be made exactly self-supporting, with no surplus, the amount of the original bond issue is always taken as

Mr. Burns. the basis on which the community must pay interest, assuming that the sinking fund with which to pay off the bonds is based on an expectancy equal to the life of the property; that is to say, if the bond issue runs for the number of years represented by the life of the property, the community must set aside a sinking fund sufficient to pay off the bonded indebtedness, and must likewise continue to pay interest on the full amount of the bond issue, regardless of the fact that the property itself is continually depreciating in value. In fact, this method of financing the enterprise is absolutely essential if the project is to be made self-sustaining. How, then, can there be any escape from the conclusion that a privately owned utility must receive its return on the same basis, for to receive less will undermine the integrity of the investment, and is, therefore, not a fair return. Of course, if the municipal bond issue is to be paid serially, then the amounts paid off from time to time would be deducted from the capital on which the public must continue to pay interest, but in order to pay these bonds serially, the annual levy or contribution for sinking-fund purposes must be largely in excess of the amount necessary for the application of the sinking-fund theory. It would seem, therefore, that the proper deduction to make from reproduction cost is simply the amount by which the actual depreciation allowance exceeds the theoretical accumulation in a sinking-fund account, computed on the age and life of the property. If depreciation is figured on the sinking-fund basis only, therefore, no deduction should be made from reproduction cost.

The Public Utilities Act of California, in effect March 23d, 1913, provides, in Section 49, that:

"The Commission shall have power, after hearing, to require any or all public utilities to carry a proper and adequate depreciation account in accordance with such rules, regulations and forms of account as the Commission may prescribe. The Commission may, from time to time, ascertain and determine and by order fix the proper and adequate rates of depreciation of the several classes of property of each public utility. Each public utility shall conform its depreciation accounts to the rates so ascertained, determined and fixed, and shall set aside the moneys so provided for out of earnings and carry the same in a depreciation fund and expend such fund only for such purposes and under such rules and regulations, both as to original expenditure and subsequent replacement as the Commission may prescribe. The income from the investments of moneys in such fund shall likewise be carried in such fund."

With the administration of the depreciation fund in accordance with this Section of the California Act, it would seem that such fund must be kept separate and set aside, and as far as the owner of the utility is concerned, he has no discretion whatever over its disbursement, and he cannot withdraw such fund for any purpose other than for the replacement of depreciated property. Therefore, depreciation

cannot equitably be deducted from the cost of reproduction in California rate cases when that method is used to determine the value of the property. Mr. Burns.

Section 47 of the Public Utilities Act of Idaho, and also Section 79 of the Public Utilities Act of Missouri, use almost exactly the same language as the California Section already quoted. This indicates the tendency in recent legislation to require public utility companies to set aside sinking-fund accounts, or, in other words, it takes these accounts out of the hands of the public utility owners and leaves them largely under the control of the public through its representatives, the public service commissions. There can be no objection to leaving the collection and disbursement of the sinking fund entirely under public control through proper utility commissions, and, in fact, it is highly desirable that such should be the case, but it is important that the commission should study carefully the conditions surrounding each property in order that the amount of depreciation collected from the rate-payers, and thus set aside, shall be adequate and equitable under all the circumstances.

Contingent depreciation should receive special consideration in each case, its amount depending on the nature of the property. Contingent depreciation is the sum that should be provided from earnings to meet unusual or unexpected replacements which become necessary from time to time in the experience of every public utility, such, for example, as damage from floods, destruction of property by earthquake or ice gorge, failure of water supply, and many similar experiences. Depreciation by obsolescence or changes in the art should also be fully covered by the allowance for contingent depreciation. In fact, contingent depreciation may be defined as the sum that should be provided for insurance against such contingencies as cannot be covered by any class of commercial insurance. This raises an important question: Is the sinking-fund method of computing the depreciation fair to the owner of a utility in a rate-regulation investigation? Evidently not, unless the rates are based on the cost of reproduction undiminished by depreciation, but, under the usual procedure in a rate investigation, it is extremely difficult, if not utterly impossible, to receive consideration for the full reproduction value of a public utility, and would it not be more practical, therefore, to adopt some system of accounting depreciation that will secure equity to both parties when considered in conjunction with a proper rate of return on the remaining expectancy of the property? Perhaps this can be understood more clearly by considering the property as passing through a series of successive ownerships. It cannot be denied that the owner may sell the property at any time and retain the accumulated depreciation fund. This fund is due him, for, if he sells the property at its fair value, computed

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on the sinking-fund basis, he must retain the depreciation fund in order to recover the full amount of his investment.

Consider again the case in its simplest form, the gravity system of water-works previously cited, and, for the purpose of simplicity, assume that no extensions or betterments are required throughout the 20 years' life of the property, and that no elements of appreciation in value accrue to affect any portion of the works. The first owner should receive from the rate-payers the sum of $3 229 for depreciation and a reasonable return on his investment of $100 000. He sells the property at the end of the first year at its value, computed on the sinking-fund basis, namely, $96 771. The second owner should receive from the rate-payers for depreciation the sum of 3.475% of his investment of $96 771, or $3 362.79—this being computed on the basis of the remaining life of the property, 19 years—and, in addition thereto, he should receive a reasonable rate of return on the reduced capital account, $96 771. Now, if such a rate is equitable to each of the successive owners of the property, it must be equally fair to the public and to the owner throughout the life of the property, if the same ownership continues throughout the entire period. For the purpose of illustration, Table 3 is submitted, computed on the basis of 4% compound interest on the depreciation fund, 8% for interest and profit on the capital investment, and a variable allowance for contingent depreciation, starting at 2% of the capital account and increasing as the property becomes older, the amount of increase being such as to preserve a constant total contribution from the rate-payers. It is desirable, from the point of public expediency, that rates be not fluctuating, and this may be accomplished by the variable contingent depreciation, based on an amount that shall represent, as nearly as can be predetermined, the facts in each case under consideration.

An examination of the column in Table 3 headed "Contingent Depreciation", shows that the variation in contingent depreciation from year to year is not great, under this system of accounting, and represents very closely what may be reasonably expected, increasing gradually with the age of the property. The contingent depreciation is not a cumulative fund, but, on the contrary, represents the amount actually spent from time to time in replacing obsolete or abandoned property and in repairing damage from unexpected or unusual causes. If properly proportioned, such a fund will fluctuate between surplus and deficit, but will maintain a general average near the zero line. The actual figures used in Table 3 are, perhaps, not appropriate for a property of this nature, but are submitted simply to illustrate a principle and without reference to the adequacy, or otherwise, as this can be determined only by a study of all the circumstances surrounding each particular property. The table is based on annual contributions to a sinking fund at the beginning of each year, although many

authorities prefer using the tables computed for the end of the year. Rates are usually collected quarterly or monthly and, therefore, as a matter of fact, it would more nearly accord with correct theory to use tables computed for annual contributions in the mid-year, if such tables were at hand. However, this difference is not sufficient to cause material error in any event. Mr. Burns.

TABLE 3.

Owner.	Capital invested.	Depreciation.		Interest and profit. 8 %.	Contingent depreciation.	Annual cost to rate-payers.
		Rate. Per cent.	Amount.			
1st.....	$100 000	3.229	$3 229	$8 000	$2 000	$13 229
2d.....	96 771	3.475	3 363	7 742	2 124	13 229
3d.....	93 408	3.749	3 502	7 473	2 254	13 229
4th.....	89 906	4.058	3 648	7 193	2 388	13 229
5th.....	86 258	4.406	3 801	6 901	2 527	13 229
6th.....	82 457	4.802	3 960	6 597	2 672	13 229
7th.....	78 497	5.257	4 127	6 280	2 822	13 229
8th.....	74 370	5.783	4 301	5 950	2 978	13 229
9th.....	70 069	6.399	4 484	5 606	3 139	13 229
10th.....	65 585	7.130	4 676	5 247	3 306	13 229
11th.....	60 909	8.009	4 878	4 873	3 478	13 229
12th.....	56 031	9.086	5 091	4 482	3 656	13 229
13th.....	50 940	10.435	5 316	4 075	3 838	13 229
14th.....	45 264	12.174	5 554	3 621	4 054	13 229
15th.....	40 070	14.496	5 809	3 206	4 214	13 229
16th.....	34 261	17.753	6 082	2 741	4 406	13 229
17th.....	28 179	22.643	6 381	2 254	4 594	13 229
18th.....	21 798	30.803	6 714	1 744	4 771	13 229
19th.....	15 084	47.134	7 110	1 207	4 912	13 229
20th.....	7 974	96.154	7 667	638	4 924	13 229

Unfortunately, in actual practice, the problem is never so simple as the gravity water-works system used in this illustration. Public utility enterprises are, from the very necessity of the case, almost continuously extended from year to year, so that the average age of a property of this nature never even remotely approaches that of its oldest parts.

If the theory outlined herein is accepted as a correct method of computing depreciation, its practical application would be somewhat as follows:

The appraiser would determine the composite life of the utility under consideration, which, for illustration, in the case of a water-works system, in some particular case might be found to be 60 years. The average age of the entire property, computed in like manner, might be found, for example, to be 20 years. The rate to be used in determining the amount of ordinary depreciation therefor would be based on an expectancy of 40 years. On a 3% sinking-fund basis, this would be 1.288%—this percentage to be applied to the remaining capital determined by deducting the sinking-fund accumulation of 20 years from the cost of reproduction of the property. The proper

Mr. Burns. additional amount to be allowed for contingent depreciation, of course, should be determined by the particular circumstances in the case. It would be more precise to compute the result separately for each group of items having the same age and estimated life; but such a procedure would, perhaps, be impractical in most cases, owing to its difficulty of application.

Mr. Knox. STUART K. KNOX, M. AM. SOC. C. E. (by letter).—Mr. Alvord's paper, dealing with "the general relation of depreciation and its effects on the fair return of a utility property," will have justified itself if it does no more than precipitate the extended discussion of this interesting and intricate subject, concerning which ideas are at present so confused and conflicting. In this, it appears to the writer, will lie its chief, if not its only, value.

As the author implies, there exists at the present time a wide diversity of opinion among engineers, lawyers, special masters, commissions, and Courts having to do with valuations for rate-making purposes, with regard to the fundamental question of the value on which fair rates should be predicated.

Among those who have sought to sustain the reproduction-cost-new theory may be mentioned the special master,* *re* Columbus Railway and Light Company *vs.* City of Columbus, Ohio, 1906; the Wisconsin Railroad Commission,† *re* City of Whitewater *v.* Whitewater Electric Light Company, decided December 16th, 1910; Massachusetts Joint Board,‡ Massachusetts Appraisal of the New York, New Haven, and Hartford Railroad, 1911; C. E. Grunsky,§ M. Am. Soc. C. E., in his paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates", to which paper the author refers; and Alexander C. Humphreys, M. Am. Soc. C. E.,|| President of the Stevens Institute of Technology, in his paper entitled "Depreciation: Estimated and Actual", presented before the Institution of Gas Engineers of Great Britain, 1913.

The contrary view, to the effect that fair rates should be adjusted to yield a fair return only on depreciated reproduction-cost, has been upheld by the Oklahoma Supreme Court,¶ *re* Pioneer Telephone and Telegraph Company *v.* Westenhaver, decided January 10th, 1911; by the New York Public Service Commission for the First District,** *re* Metropolitan Street Railway Reorganization, decided February 27th, 1912; and by the United States Supreme Court.

Mr. Alvord's ideas on the subject are summarized in his first and second "conclusions," as follows:

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* Robert H. Whitten, "Valuation of Public Service Corporations," p. 368.
† *Loc. cit.*, p. 366.
‡ *Loc. cit.*, p. 363.
§ *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.
|| *Transactions*, Institution of Gas Engineers (Great Britain), 1913, p. 348.
¶ Robert H. Whitten, "Valuation of Public Service Corporations," p. 375.
** *Loc. cit.*, p. 376.

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"1st.—That if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property 'used and useful for the public'.

"2d.—That if these funds are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope, as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain."

The writer is not prepared to admit the truth of either of these propositions without material qualification, and in the following discussion will seek to show:

(*a*) That the disposition made of the depreciation reserves has theoretically no bearing whatever on the question of value on which the owner of a utility property is entitled to earn a fair return.

(*b*) That the question of value on which an owner is entitled to earn a fair return depends solely on whether the depreciation reserve provided has been or is to be just "adequate", "excessive" in amount, or "inadequate" for the purpose intended.

He will endeavor to demonstrate that when the depreciation reserves, which have been and are to be provided, are just "adequate" for the purpose intended, the owner must, if he is to be protected against the impairment of his capital or of its earning power, be permitted to earn, in addition to operating expenses and this "adequate" depreciation reserve, a fair return on reproduction-cost-new.

That when the depreciation reserves, which are to be provided, are "excessive", so as virtually to effect a refund to the owner of a portion of his original investment, together with its full earning power, rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new, the measure of the reduction being the total amount of the capital so refunded.

That when the chosen method of providing for future depreciation is such as virtually to effect a progressive refund, free and clear, of a portion of the owner's original investment, a corresponding progressive decrease in the value on which he is allowed to earn a fair return may be equitably made.

That when the depreciation reserves which have been provided are "excessive," so as virtually to have effected a refund of a portion of the owner's original capital, rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new only if no change in ownership has taken place.

That when the depreciation reserves which have been provided are "inadequate", so that an actual impairment in capital has taken place,

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we should, to be consistent, and provided full ownership is still retained by the owner who suffered the loss, permit the adjustment of rates to yield a fair return on a value greater than reproduction-cost-new until this loss has been recouped.

That rates may be equitably adjusted to yield a fair return on depreciated reproduction cost, only if the virtual refund of capital, resulting from "excessive" past or prospective depreciation reserves, just equals the accrued depreciation on the property on the date of the valuation for rate-making purposes.

As a preliminary to the intelligent discussion of the preceding propositions, it is essential first to define exactly what is meant by "adequate", "excessive", and "inadequate" depreciation reserves, and by the expressions "accrued depreciation" and "depreciated reproduction cost".

By "adequate" depreciation reserve is meant the annuity or percentage of reproduction-cost-new which, if set aside in the bank or in trust from the beginning of operation, at the rate of interest, say 4%, on which the fund payments are predicated, will, with all interest accretions, amortize a sum which, at the termination of the useful life of each element of the property, will just suffice to replace it and no more.

By "excessive" depreciation reserve the writer means an annuity greater in amount, and by "inadequate" depreciation reserve an annuity less in amount than the above.

By "accrued depreciation" at any period of the life of a property, is meant the amount which an "adequate" depreciation reserve would have amortized had it been set aside each year from the beginning of operation.

By "depreciated reproduction cost", is meant the reproduction-cost-new less the "accrued depreciation" so determined.

By the "useful life" of any element of property, is meant its age plus the remaining life expectancy of the element.

In explanation and justification of the foregoing definitions, it may be well to digress at this point to the extent of acknowledging a full realization of the fact that any estimate of accrued or prospective depreciation is at best a guess. There are, however, intelligent guesses and unintelligent guesses, and there are intelligent ways of guessing and unintelligent ways of guessing. One unintelligent way of guessing the accrued depreciation on a pumping engine, for example, is to sit in the office and compute it, starting with the assumption that the normal life expectancy of a new unit of the type considered is 20 years, and that the age of the particular unit under consideration is, say, 10 years. The unit may have been in continuous service from the date of its installation, or it may have been used largely as a spare. It may have been operated 24 hours per day, or only 1 or 2 hours per

day. It may or may not have had intelligent care, and it may or may not be well adapted to the service, well designed, and of modern type. No intelligent guess of accrued or prospective depreciation can be made without a visual field inspection of the element to be appraised. Mr. Knox.

Whatever be the age of a unit, or structure, the present value of which it is desired to ascertain, the uncertain element affecting this value which may be guessed most accurately by an expert is its remaining life expectancy. In the light of the history and probable future of the plant, of the community which it serves, and of the arts, this, after an examination of the unit or structure, may be gauged as closely as any other probability affecting the future of the property. Then, if the guess proves accurate, the "useful life" of the element becomes its age plus its life expectancy on the date of the appraisal; and an "adequate" allowance to cover annual depreciation is the annuity, which, had it been paid into a fund each year since the installation or construction of the unit or structure, would have amortized a sum sufficient to reproduce it at the end of this "useful life". Under these conditions the "accrued depreciation" on the date of the appraisal may fairly be estimated as the amount which such an annuity would have accumulated between the date of installation and the date of the appraisal, and its fair market value or depreciated reproduction-cost at the date of appraisal may fairly be assumed to be its reproduction-cost-new less the accrued depreciation so found. Fair market value under these circumstances would of course be defined to be the value as established between a seller who was willing but not compelled to sell and a buyer who was willing but not compelled to buy.

In all that follows, it will be assumed that the guesses as to life expectancy and hence of proper or "adequate" depreciation reserves are accurate ones.

To come to the discussion of Mr. Alvord's conclusions, it is undoubtedly true as the author states that "there must be in this, as in all other questions relating to this difficult subject of valuation, a just and equitable relation between the public and the public utility owners, which is founded on rational analysis and common sense." In seeking out this relation much is gained in the way of clarity by confining ourselves at first to the consideration of simple examples. We are thus able to eliminate the many complicating, confusing, and often entirely irrelevant considerations which usually cloud and obscure the vital principles involved, when the question is studied in the light of complex cases such as commonly come before the Courts. Although it is no doubt true, as the author implies, that conclusions deduced from the consideration of simple examples will not always admit of indefinite extension to cover the complex cases met in actual practice, we seem at least to be on safe ground in assuming, as a point of departure, that a conclusion which will not withstand the most searching

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analysis, when viewed in the light of its application to a simple practical example, cannot be the basic principle or relation which we are seeking. At any rate, let us grant the truth of this proposition for the moment, and observe the result of applying such a test to the conclusions reached by Mr. Alvord, beginning with his first conclusion, "that if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property 'used and useful for the public'."

If by a "properly computed and accurately kept" sinking fund, Mr. Alvord means an "adequate" depreciation reserve, as defined in a preceding paragraph of this discussion, the logic of this conclusion appears to be unassailable.

Consider, for example, a water-works plant, consisting solely of a pipe line 10 miles long. The owner of this property, we will say, buys water at wholesale at one end of the pipe line, and sells it at wholesale at the other end. The writer has in mind a private water company the property of which is substantially as described. Assume that the reproduction-cost-new of the pipe is $1 000 000, its useful life 50 years, and its scrap value nothing. No part of the line will require replacement during its 50-year life, and the pipe may be maintained in good serviceable condition throughout the entire period by making such ordinary repairs as may be properly charged to operating expense. Neglecting the fact that the creation of a sinking fund must lag one year behind the commencement of operation, the proper annual reserve to cover depreciation, estimated on a 4% sinking-fund basis, is $6 550, which amount the owner places in bank each year at 4% compound interest.

In the construction of this pipe line, the owner converted $1 000 000 of money into physical plant. As the pipe ages, it depreciates in value, and if the guess as to its useful life (50 years) proves correct, the amount of this depreciation is, as nearly as may be, represented at any time by the accumulation in the sinking fund or reserve fund for depreciation. The process may be regarded as the gradual re-conversion of plant value into cash, until at the end of the pipe's useful life the owner has all cash and no pipe line, at which time the money accumulated is again converted into physical plant through the taking over of a new pipe line which the owner has caused to be constructed. At any intermediate period of the life of the pipe, the owner's capital is represented partly by the depreciated value of the line, and partly by the depreciation reserve. The sum of the two may be considered as representing at any time the reproduction-cost-new. A purchaser can at any time afford to pay for the pipe line and depreciation fund together, a sum equal to the pipe's repro-

duction-cost-new, for by continuing the depreciation reserve in bank, and adding the proper annual payments, as these are earned, he will have available at the termination of the useful life of the property a sum equal to his original investment and to the pipe's reproduction-cost-new. This is equivalent to saying that a purchaser can at any time afford to pay for the pipe line alone a sum equal to its reproduction-cost-new less the accumulations in the sinking fund, or its depreciated reproduction cost. Mr. Knox.

Now, as all interest accretions, if the annual depreciation reserve is a proper or "adequate" reserve computed on the sinking-fund basis, must be credited to the fund, the owner enjoys no income therefrom, and, therefore, should be allowed to earn a fair return on the depreciated physical value plus the depreciation reserve, or on the reproduction-cost-new. To assert the contrary is to assert that the owner must, in spite of everything he can do, suffer a financial loss through impairment of the earning power of the capital which he has invested.

Let us now examine Mr. Alvord's second conclusion:

"That if these funds [adequate depreciation reserves] are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope, as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain."

Suppose that the owner of the pipe line operates his plant in exactly the same way as described, except that instead of actually placing $6 550 in bank or in trust each year as part of the property, he invests this amount in other ways, or even places it in a safe deposit vault or squanders it. Mr. Alvord concludes that under these circumstances rates may be equitably adjusted to yield a fair return only on the depreciated physical value of the pipe instead of on its reproduction-cost-new. This means that unless the annual depreciation reserves are correspondingly increased above the "adequate" amount previously estimated, a sliding scale of rates must be introduced, such that the amount available for interest in the tenth year of operation, after deducting operating expenses and depreciation reserve, will be approximately 92%, in the fortieth year approximately 38%, etc.. of its amount in the first year of operation. As the service is now by hypothesis exactly the same as in the case where the depreciation reserve was actually deposited in bank as part of the property, the water consumer will evidently gain in reduced rates by the owner's failure to establish an actual depreciation reserve. It is equally evident that the consumer cannot gain unless at the same time the owner loses an equal amount. Let us pursue the illustration further.

Suppose that the owner, in the tenth year of operation, becomes alarmed at the dwindling profits on his water transportation business,

Mr. Knox. and begins to busy himself in devising a plan for restoring rates to the point where they will yield the same return on his capital as in the first year. He estimates that if he had from the outset maintained a depreciation fund in the bank at 4%, the accumulations in this fund would at the end of the tenth year have amounted to $78 760.56. He, therefore, sells at par from his private means seventy-nine $1 000 municipal bonds drawing 4% interest, or he sells, we will say, an apartment house valued at $78 760.56, which has been yielding him a net return of 4% per annum. He deposits the proceeds, $78 760.56, in the bank at the beginning of the eleventh year and christens it a depreciation fund. He has not lost a dollar by this transaction. The depreciation fund is his property in the same way that the house was his property. The interest yield is the same. He has simply performed the familiar operation of taking a sum of money out of one pocket and placing it in another, and yet, as a result of this simple shifting of assets, he has placed himself in a position where he may equitably charge his customer, the water consumer, we will say, an additional $10 per 1 000 000 gal. for exactly the same quantity and quality of water, delivered under exactly the same pressure as before the depreciation fund was created. Could anything be more absurd?

Mr. Alvord speaks of the use of the depreciation reserves, by the owner, "for other personal gain". It is, of course, apparent that when these reserves are proper or "adequate" reserves computed on the sinking-fund basis, it is entirely beyond the power of the owner to use them so as to yield "a private gain", unless he invests them in a way to yield a return greater than that which they would have earned if placed in the bank or in trust at the rate of interest on which the sinking or depreciation fund payments were computed.

If correctly computed on the sinking-fund basis, the sum of the annual reserves will, by hypothesis, produce an amount sufficient to provide for future replacements, only if credited with all interest accretions resulting from their conservative investment. The source of these interest accretions is immaterial. The funds are equally unproductive, as far as concerns their power to yield a "private gain" to the owner, whether they be placed in bank at 4% interest, invested in high-grade municipal bonds at 4% interest, or invested in other ways. If the owner is fortunate enough to invest them in a way to yield a return greater than they would have earned if placed in a bank or in trust, his "private gain" is represented, not by the total yield of the funds so invested, but only by the excess of this yield over and above what they would have earned if placed in the bank or in trust. To obtain this excess yield or "private gain", the owner performs services and assumes risks for which he is entitled to compensation represented by the excess yield. He has, in fact, entered the banking business and is as much entitled to a fair profit on this banking business

as he is on his water-works business. In his capacity of water-works manager, he deposits with himself, in his capacity of banker, the depreciation reserves on which he pays 4% compound interest. When the time arrives, he must return these sums with accumulated interest, the same as any other banker would be obliged to do. In the meantime he is entitled to invest them conservatively in such a way that they will yield enough more than 4% to compensate him for performing the services and assuming the responsibilities and risks which attend the banking business. Mr. Knox.

There are bankers and trust companies, many of them, that actually own and operate water companies. Is it to be presumed that these banking institutions, to avoid being "fined," must invest their depreciation reserves with some competitor, perhaps less responsible and competent than themselves? Again, the proposition is absurd.

Theoretically, the consumer has no interest whatever in the depreciation reserves or in the disposition which is made of them. He is interested only in the quality of the service. Service is what he pays for. He pays not for the quality of service which an element of plant is capable of rendering when new, or at its absolute maximum of efficiency, but for the average service which it is capable of rendering during the period known as its "useful life," when maintained throughout this life in the highest state of efficiency which is physically possible, and replaced at its termination with a new element. In the absence of certain knowledge to the contrary, it must be presumed that the owners will so maintain and so replace the plant, regardless of whether or not an actual, tangible depreciation fund is maintained in a bank—a competitor's bank. Even when depreciation reserves have been distributed to stockholders, or lost or tied up in unfortunate investments, it must be presumed, in the absence of certain knowledge to the contrary, that the owners will provide funds as required by dividend reductions, assessments against the stock, or in extreme cases by reductions in interest.

As a practical matter, it is granted that in certain cases the owner's failure to invest depreciation reserves conservatively may react on the consumer, and that to this extent he is interested in the disposition which is made of them. It is, for example, a matter of common observation and knowledge that when an owner first becomes involved in financial difficulties there exists the temptation and tendency to maintain dividends and interest at the expense of maintenance and replacements. The prevention of this is one of the functions of the regulating power.

As long as the quality of the service is actually maintained, there can exist no logical reason why the value on which the owner of a property is entitled to earn a fair return should be variable, depending on whether or not a reserve for depreciation is maintained in the

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bank or in trust as part of the property. To penalize or "fine" an owner for failure to maintain an actual fund by adjusting rates to yield a fair return only on depreciated reproduction-cost, is to presume in advance of the fact that he will not, when required, and at his own cost, make the replacements for which depreciation reserves were provided. It is in effect to punish him in advance for an offense that it is suspected he may commit at some time in the future.

The points just made have been clearly stated by the Wisconsin Railroad Commission *re* City of Whitewater and Whitewater Electric Light Company, decided December 16th, 1910.* The Wisconsin Railroad Commission says:

"As it is a general rule that the reasonable return which a utility is allowed to earn covers the interest and depreciation on the actual investment in the plant, it becomes important to know what the investment in the plant actually is—that is, what is the value of the plant new. The fact that the property of the utility has diminished in value with use, as the inevitable result of depreciation, does not lessen the amount of the investment in the plant. To be sure, it may happen in the case of a given utility that money which should have gone to the establishment of a depreciation fund has been diverted to the stockholders, thereby apparently lessening their investment. If an amount equal to the difference between the value of the plant new, and the value in present condition is thus paid over to stockholders, it would appear, at first sight, that the value of the plant in present condition would be the basis on which interest returns should be allowed. But it must not be forgotten that at the expiration of the life of the plant if the money which should have been used to provide for depreciation has been paid to stockholders in the form of dividends or otherwise, the value of the plant will be nothing. Then, instead of the utility having a depreciation fund on which to draw to replace the plant, the owners will find it necessary to pay the cost of replacement, presumably from the money which they have received from the plant, but which should have been used to provide a depreciation fund. The investment in the plant then must, in general, be taken at the cost of the plant new, since although the investment may apparently be diminished by failure to provide for depreciation, and by the payment of this money to owners or stockholders, in reality the investment is not diminished, because of the necessity of replacing the plant, in the absence of a depreciation fund, from the property of owners or stockholders. Therefore, it appears that the question of valuation, which is of most importance in the case, is that of cost of the plant new, or the actual value of the total investment in the plant."

It is not necessary to state that a reduction, below reproduction-cost-new, of the value on which an owner is permitted to earn a fair return, may be logically justified only on the assumption that a virtual refund of a portion of the capital originally invested has taken place prior to the date of the valuation for rate-making purposes. It has

* Robert H. Whitten, "Valuation of Public Utility Properties," p. 366.

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been shown that no such refund is effected where an owner, in lieu of maintaining an actual tangible depreciation fund in the bank, invests these reserves in other ways. There are, however, ways in which such a virtual refund may occur, and it may prove interesting to examine one or two of them.

Suppose, for example, that the owner of the pipe line, previously referred to, instead of setting aside each year in addition to operating expenses and interest an "adequate" depreciation reserve of $6 550, sets aside a depreciation reserve, improperly computed on the straight-line basis, of $20 000 per year. Under these circumstances interest accretions are no longer, of necessity, credited to the fund, for the sum of the annual payments themselves, will, without interest, accumulate an amount sufficient to replace the pipe line at the end of its useful life. Whether placed in the bank as an actual tangible depreciation fund, or invested in other ways, the owner may each year draw and add to his gross earnings the income from this fund without in any way defeating the object for which it was created. However it be invested, the entire fund may now be used productively by the owner "in private gain." A portion of his original investment equal in amount to the sum of the annual depreciation reserves has in effect been refunded to the owner by the consumer. It has been refunded free and clear, together with its full earning power, and if the owner is to be prevented from receiving, at the expense of the consumer, a double interest return on the same principal, rates must be adjusted to yield a fair return not on reproduction-cost-new and not on depreciated reproduction cost, but on reproduction-cost-new less the sum of the "excessive" annual depreciation reserves. This value will be less than the depreciated reproduction cost. At certain periods during the life of the pipe, it will be materially less. At the end of the twenty-fifth year, for example, the "accrued depreciation," or the sum amortized by an "adequate" depreciation reserve of $6 550 per annum, will be only $272 803, whereas the sum of the annual depreciation reserves, without accrued interest, will be $500 000. The depreciated reproduction cost on that date will be $1 000 000 — $272 803 = $727 197, whereas the value on which the owner should be allowed to earn a fair return will be only $1 000 000 — $500 000 = $500 000, or about two-thirds of the depreciated reproduction cost.

If an owner has, during the past, set aside out of earnings an "excessive" annual depreciation reserve, or if he has, what amounts to the same thing, enjoyed excessive profits, it may appear from the preceding discussion that the ends of justice will be served by adjusting rates to yield a fair return on a value less than reproduction-cost-new. The justice of the procedure, when adopted as a retroactive or punitive measure, is, however, only apparent. The ownership of most public utility properties is constantly changing through transfers of stock,

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and reprisal would in many cases fall not on the original owners, who alone could be supposed to merit it, but on the present owners who purchased in good faith, at a fair price, and reaped no benefit from past extortions.

Suppose, for example, that at the end of the twenty-fifth year, and just prior to the date of a valuation for rate-making purposes, the pipe line, previously referred to, had been sold by the original owner, A, to the present owner, B. Suppose that A, throughout the period during which he operated the line, had extorted from the public, in addition to a fair return on his capital, improper depreciation reserves estimated on the straight-line basis, which amounts he did not transfer to B with the depreciated property. A has now placed himself beyond the reach of reprisal. He has gone out of the waterworks business, taking his unholy profits with him. He has sold his plant for its market value which, for the purposes of this discussion, may be assumed to be fairly represented by its depreciated reproduction cost at the date of sale ($727 197). He could not have sold it for more however moderate his early charges for depreciation had been. B has purchased the property in good faith and for a fair price. It might now be argued superficially that as far as B is concerned, the price paid represents his total investment, and that the adjustment of rates to yield a fair return on this purchase price will impose no hardship on him; but this is not the case. The fallacy in the reasoning, though perhaps not obvious, lies in the assumption that the money which B has paid to A for the physical property represents his total investment in plant. It does not. A has sold to B a depreciated physical property plus a liability. The liability consists in the obligation which B incurs of replacing the depreciated pipe line at the end of its useful life. To place himself in a position to meet this obligation when it matures, B must, on the date of the purchase, set aside in bank at 4% interest an amount ($272 803) equal to the accrued depreciation on the plant. If he fails to do this, he, nevertheless, automatically, by the assumption of the obligation, ties up an equal amount of his outside investments as far as concerns their power to yield him a "private gain." In any case his total investment is not represented by the sum of money paid to A, but by this amount plus the accrued depreciation. Whatever disposition he makes of the depreciation reserve, if it is only an "adequate" reserve as previously defined, B will inevitably suffer a financial loss unless allowed to earn a fair return on $1 000 000, the reproduction-cost-new. Any reduction in this value will inevitably work a great injustice by causing B to suffer for the sins of his predecessor.

B, it is true, will be protected against loss, if at the same time that rates are adjusted to yield a fair return on depreciated reproduction cost, he is allowed to set aside each year from earnings, an "exces-

sive" depreciation reserve sufficient to reproduce the depreciated plant value ($727 197) at the end of its 25 years of remaining life. It may indeed be laid down as a general rule that where past depreciation reserves with interest accretions have been adequate to take care of the accrued depreciation, and future reserves are made sufficient in themselves to amortize sums which will provide against all subsequent deterioration, rates may fairly be predicated on the depreciated reproduction cost at the date of the valuation for rate-making purposes. Mr. Knox.

Although there is one notable exception which the writer will discuss later, the adoption of this rule will usually result in difficulties and confusion in accounting, with no attendant benefits either to the consumer or to the owner. Decreasing the amount allowed for interest and at the same time allowing a corresponding increase in amounts allowed to offset depreciation is a mere juggling of figures. An annuity sufficient to amortize the depreciated value of a property at the termination of its remaining life expectancy will become an "excessive" annuity on the day the first replacement is made, and generally the depreciated reproduction cost will be modified as a result of each replacement.

On the other hand, if we neglect the fluctuations in the prices of labor and materials, reproduction-cost-new will remain an unvarying quantity as long as no additions are made to the property, and the "adequate" annual depreciation reserve, estimated in the customary way, will remain at all times a constant percentage of the unvarying reproduction-cost-new. The reproduction-cost-new having once been ascertained may subsequently be kept up to date merely by adding the cost of extensions as these are made, and the "adequate" annual depreciation reserve, having once been ascertained, may be kept up to date, as extensions are made, by adding sums sufficient to reproduce the cost of each such extension at the termination of its useful life.

If past depreciation reserves, with interest accretions, have been no more than adequate to offset the accrued depreciation on the date of the valuation, either method will result in justice both to the consumer and to the owner. The choice of method is merely a question of convenience, and generally it will be found most convenient to predicate rates on reproduction-cost-new, at the same time limiting the annual depreciation reserve to the "adequate" annuity, as previously defined.

The exception to which the writer has referred is presented by the case, briefly touched on by Mr. Alvord, and also discussed by Henry Floy, M. Am. Soc. C. E.,* in which certain large properties, after suffering an initial depreciation, reach a stable stage in which they may afterward be maintained by the expenditure of certain sums each

* "Valuation of Public Utility Properties," p. 194.

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year for replacements, the cost of these replacements being charged to operating expenses the same as ordinary wear and tear. To quote Mr. Floy:

"The Receiver of the Third Avenue Railroad in New York City, operating a large property having numerous physical elements so that all deterioration became simply 'wear and tear' and a part of operating expenses, declined to obey the order of the Public Service Commission and provided no depreciation fund whatever, simply removing deterioration when it occurred and charging it as maintenance in operating expenses."

If past depreciation reserves, with interest accretions, have been no more than "adequate" to offset "accrued depreciation," all subsequent depreciation on a property which has reached this stage of stability, may properly be taken care of in either of two ways:

1st.—By continuing the "adequate" annual depreciation reserve; in which case rates must be predicated on reproduction-cost-new.

2d.—By providing annually, in the manner adopted by the Receiver of the Third Avenue Railroad, a sum sufficient in itself to take care of all subsequent depreciation, in which case rates should be predicated on depreciated reproduction cost.

By the second method the annual depreciation reserve is in excess of an "adequate" reserve, as previously defined, and it is for this reason, and for this reason alone, that rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new.

Consider, for example, a plant composed of five elements of equal value and variable life expectancy as shown in Table 4.

TABLE 4.

Element. (1)	Reproduction-cost-new. (2)	Life expectancy, in years. (3)	Annual depreciation reserve to refund reproduction-cost-new at termination of useful life, sinking-fund basis. (4)
A	$1 000	25	$24.01
B	1 000	30	17.83
C	1 000	35	13.58
D	1 000	40	10.52
E	1 000	45	8.26
Total, or average...	$5 000	.....	$74.20

Assuming the scrap value to be nothing, the orthodox way of providing for the future depreciation on such a plant would be to set aside each year from the beginning of operation sums sufficient to amortize the reproduction-cost-new of each element at the end of its

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useful life. The annuities so required, estimated on a 4% sinking-fund basis, are given in Column 4 of Table 4. The total annuity required is $74.20. At the end of the twenty-fifth year, element *A* must be replaced, and the cost will be met by withdrawing from the depreciation fund the $1 000 accumulated by the annuity of $24.01 provided for this purpose; but, as the new element, *A*, will also have a life expectancy of twenty-five years, at the end of which it must again be replaced, the annuity of $24.01 provided to cover depreciation on element *A* must be continued and the total annuity will remain $74.20 as before. It will remain so indefinitely as long as no additions are made to the plant. Whether it be placed in bank or in trust at 4% interest, or invested in other ways, the annuity of $74.20 will, together with all the interest accretions with which it may fairly be credited, just take care of future depreciation, and no more.

Assuming that the owner actually makes such an annual reserve, the amount amortized at the end of the twentieth year will be $2 209.65, which amount may be taken to represent the accrued depreciation on that date. The depreciated reproduction cost at the end of the twentieth year will then be $5 000.00 — $2 209.65 = $2 790.35. As the owner can by hypothesis derive no "private gain" from the depreciation fund, however it may be invested, he must, so long as he provides for depreciation in this way, be allowed to earn a fair return on reproduction-cost-new.

Now assume that at the end of the twentieth year the owner perceives that 5 years hence he must replace element *A* at a cost of $1 000. He realizes further that at the end of each succeeding 5-year interval he must replace one element of his plant at a cost of $1 000. He estimates that the annuity necessary at 4% compound interest to produce $1 000 at the end of 5 years is $184.63, and he elects to take care of all subsequent depreciation by reserving this amount each year from earnings. The future cost to the consumer under this scheme will be increased in the annual sum of $184.63 — $74.20 = $110.43, and he must, obviously, if he is to avoid loss, obtain a reduction in the plant value or capitalization on which the owner is allowed to earn a fair return. Nor will the owner under these circumstances suffer any loss by such reduction in value. He has, in a sinking fund or otherwise invested, at the end of the twentieth year $2 209.65, equal to the accrued depreciation on his property as of that date. Under the first or orthodox method of providing for replacements, this sum, together with its earning power, was necessary to the proper provision for depreciation. Under the second scheme, neither the sum nor its earning power is so required. On the date on which the owner changes over from the first to the second method of providing for future depreciation, this fund is released or set free, and may henceforth be used productively by the owner in any way he sees fit. The consumer has in effect, by permit-

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ting the change in method, refunded to the owner, free and clear, a portion of his original capital equal in amount to the accrued depreciation on the property. Henceforth the owner's investment in plant is equal, not to the reproduction-cost-new, but to the depreciated reproduction cost, and this therefore is the sum on which he should be allowed to earn.

The case presented is only a special one under a general rule in which the interests of convenience are served by lowering below reproduction-cost-new the value on which the owner is allowed to earn a fair return, at the same time increasing above the "adequate" depreciation allowance, the sums annually set aside out of earnings to offset deterioration. It is in no sense an exception to the general law that where depreciation reserves are only "adequate" reserves, as previously defined, rates must be predicated on reproduction-cost-new.

The application of the general law will usually be more convenient, and will always result in justice both to the public and to the owners, unless we regard as an exception the case in which the owner has actually, during the past, set aside from earnings as an offset to deterioration an annual sum more than sufficient to take care of the depreciation which has occurred, or, what amounts to the same thing, unless the owner has actually during the past enjoyed excessive profits for which he is now to be penalized.

A reduction in the value on which he is now permitted to earn may be logically justified on this ground only in the event that the present owner has benefited from these extortions. Even on this supposition, the logical and scientific procedure is to take as our point of departure the basic principle that in the absence of such complicating considerations, the owner should be allowed to earn on reproduction-cost-new; then to apply proper and specific allowances for such irregularities as the history of the plant may disclose. In this way "the punishment" may be made "to fit the crime," and we will always know, at least by how much and why, we have lowered or increased below or above reproduction-cost-new the value on which the owner is entitled to earn a fair return, for it must not be forgotten that it is a poor rule which does not work both ways, and if an owner is to be penalized for "excessive" depreciation reserves or exorbitant profits made in past years, consistency demands that he be compensated for his past failure to provide "adequate" depreciation reserves and fair profits.

If, for example, the owner in the case just discussed, has during the first 20 years, in which the necessity for replacements was not conspicuous, made no reservations whatever for depreciation, but has been satisfied to earn a fair return on his investment in addition to operating expenses, in the expectation that future replacements may be provided for as a part of operating expenses, when the occasion arises it is evident that his capital will be impaired by an amount ($2 209.65) equal

to the accrued depreciation if rates are adjusted in the twentieth year to yield a fair return only on depreciated reproduction cost. In this instance, however, the loss is largely the result of his own folly, in failing to make provision at the proper time for the deterioration which was taking place. Mr. Knox.

He would be obliged to incur the same loss if he undertook to sell the property in the twentieth year, as it cannot be presumed that the physical elements of the plant could be sold on that date for an amount greater than their depreciated reproduction cost.

Mr. Humphreys, in discussing this question in the paper to which the writer has previously referred, quotes an amusing illustration taken from what he characterizes as the most logical paper on this subject he has ever read. The quotation is credited to a brief by Charles F. Mathewson, of the New York Bar, in the case of Kings County Lighting Company *vs.* The Public Service Commission for the First District of New York. Mr. Mathewson was the trial lawyer for the Company in the New York Consolidated Gas Company's case. The quotation follows:

"The proposition [to deduct 'accrued depreciation' in valuing plants in rate-making] is so absurd on its face that it hardly needs discussion to show its fallacy. Why, aside from the question of 'confiscation,' should consumers, for exactly the same service, equally efficiently rendered, expect to pay less in the sixth year than in the first year, merely because some items of plant will (viewed at the sixth year) require replacement at a date in the future then nearer than such date was at the beginning of operation? As well might it be claimed, to repeat a homely illustration, that a farmer should regulate the price of the eggs which he sells, by the age of the hen which lays them—reducing the price of the product as the hen gets on in years. The reason he does not is that the service efficiency and operating value of the hen, as evidenced by the quality of the eggs which she lays, are not impaired by the fact that her life is advancing. That advancement may concern the farmer and possibly concerns the hen; but it in no manner affects the value of the eggs to the consumer, or justifies him in demanding them at a lower price than he paid at an earlier period of her life. The consumer of the eggs must expect to pay a sufficient price to afford a return to the farmer on his total investment in the hen during her life, *plus* enough more to enable the farmer on her death to replace her, and thus keep his investment unimpaired. A farmer could hardly be expected to invest in hens for the purpose of supplying the public with eggs, if for a portion of their life he was to receive a return on only a third or a half of his investment; and any such rule would simply compel the public to go without eggs until the regulating power (if such there were) saw fit to revise its reasoning. There is absolutely no difference in the economic principles applicable to the operation of a gas plant and the operation of a hennery, so far as concerns right to return on capital; and what is absurd in one case is equally absurd in the other. The fact that the rate of return in the one case is subject

Mr. Knox. to reasonable regulation, and not in the other case, has no bearing on the main proposition."

A consideration of this illustration in the light of the principles thus far set forth indicates that it is not of universal application except as an illustration of the fundamental principle which always holds where complicating considerations are absent. It presumes that the farmer charges no more than enough to afford a return on his total investment plus enough to keep his investment unimpaired. If, as some consumers think, the farmer charges enough in addition to a fair return on capital invested, to replace the hen several times over, prior to her demise, the absurdity of expecting a reduction in the price of eggs, even in this case, from a purely theoretical point of view, is not quite so apparent.

It is readily admitted that the preceding discussion has been carried on largely from a theoretical point of view; but, considering the present confusion of ideas on the subject, this theoretical treatment of the question appears to be fully justified. Sound practice, in this as in other engineering matters, must rest on a firm foundation of sound theory, and the establishment of this theory has something more than academic interest.

In concluding it may be well to point out that too much importance may easily be attached to the fact that the Supreme Court of the United States has in certain instances held that rates should be adjusted to yield a fair return only on depreciated reproduction cost. It is of course true that the last word in these matters rests with the Courts, and that Courts are much given to following precedent; but it must not be forgotten that Court decisions are usually rendered with a view to dispensing justice in the particular case before the Court. It has been shown that there may be circumstances in which the ends of justice are served by allowing an owner to earn a fair return on depreciated reproduction cost. Whether or not the cases so decided by the United States Supreme Court fall in this category we do not presume to say, but even this is unimportant because it cannot be presumed that the Courts will seek to perpetuate what can be demonstrated to be a wrong merely to avoid going against their own precedents. The question cannot be regarded as finally settled until it is settled equitably, and when, as in the present case, such a wide diversity of opinion prevails among those who have spoken most authoritatively on the subject, the attempt to reason the problem out *a priori* seems to be fully justified.

Mr. Gandolfo. J. H. GANDOLFO, ASSOC. M. AM. SOC. C. E. (by letter).—In a discussion dealing with appraisal, valuation, depreciation, rate-making, or kindred subjects, the argument is frequently advanced that no consideration must be given in cases in which all matters pertaining thereto have not been conducted in a fair, honest, proper, and business-

Mr. Gandolfo.

like manner. If the human race was still living in the Garden of Eden; if Pandora had never allowed all those troubles to escape into the world; if there were no such things as selfishness, greed, hate, envy, revenge, and a host of other evil passions; in short, if the great struggle for existence throughout the world (and this struggle applies to all plant and animal life, as well as to all members of the human race) did not exist, then it would be possible to discuss such questions purely academically, and arrive at such a course of conduct for all parties concerned as would lead the human race along a broad and happy highway of peace, prosperity, and contentment. Under these circumstances, the very necessity for such things as public service commissions, appraisals, hearings, judicial reviews, and hosts of kindred matters would not exist.

Unfortunately, however, the millennium is still as far away as ever. It is just such elements as those described in the first part of the foregoing paragraph that go to make up the human mind, and therefore such elements must be taken into consideration and given careful attention in such discussion. In other words, it is the much used term "human nature" that must be studied and reckoned with, and taken into account, in such investigations. Facts on record relating to bond and stock issues, and to general methods of financing and of business, especially in the railroad world, have made history, showing most conclusively that human cupidity and the still baser human passions have played a most important rôle in these matters from the year 1838 to the present day, resulting in the loss of millions of dollars to honest investors, both large and small.

Mr. Alvord says, on page 2048:* "In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand". In the succeeding pages, however, he seems to lean to the idea that a sinking fund provided to take care of depreciation or obsolescence can be invested in some other enterprise and still remain a sinking fund, and goes into a great deal of detail to explain why this is so. For example, on page 2056,* he says: "* * * for instance, such moneys may be invested as new capital in new construction and extensions in the very plant itself to great advantage * * *."

Webster defines "sinking fund" as follows: "(*Finance*), a fund created for sinking or paying a public debt, or purchasing the stock for the government". The Century Dictionary and Cyclopædia gives the following: "A fund formed by a government or corporation for the gradual 'sinking', wiping out, or reduction of its debt, by various devices for the accumulation of money." Funk and Wagnalls' Standard Dictionary says: "A fund instituted and in-

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vested in such wise that its gradual accumulations will enable it to meet and wipe out a debt at maturity".

In view, therefore, of the very fundamental idea and *raison d'être* of a sinking fund to cover depreciation and obsolescence, it is an absolute impossibility to consider it as being invested in any other enterprise, except as a bank or trust company would invest such funds. The moment such a sinking fund is used by the owner of a plant or a board of direction for any purpose other than that for which it was originally set aside, the fund, as a sinking fund, ceases absolutely to exist. No longer can it be counted on as needed to replace the plant. It is gone, absolutely and entirely, as far as its original purpose was intended, and no amount of bookkeeping or argument can make anything else out of this fact.

On page 2054* Mr. Alvord says: "* * * there would be no difficulty or hazard in relying on the ability of the owner or owners to replace the sinking fund from outside sources * * *." On page 2055* he also says, "It may be objected that it is a hardship on owners of public utilities, who are able and willing to replace in the property such portion of the renewal fund at any moment * * *", and on page 2056,* "* * * the owner must be willing and ready to finance the replacements and depreciation fund, whenever the occasion therefor arises * * *." Admitting for the moment the willingness, suppose, when the time comes, that the owner is not able to do so? What then? And further, suppose that such a fund has been invested in some other way, or has been appropriated for some other purpose, and then the parties at fault, although fully able to do so, are not willing to make restitution when the fund is needed for its original purpose. Who is to make them do so?

In connection with this matter of the investment of a sinking fund, it may be argued that the bank or trust company in which such a fund would be deposited, to draw interest at ordinary rates, would invest these moneys in some way, and therefore there is practically no difference in the end, than if the owners invested the fund direct. There is a very great difference, however, which may be summed up as follows:

(1) The owner is an interested party, and as such, is not capable of investing the fund to as good advantage as the bank. He would be more apt to have bias or prejudice, good or bad, and it would be absolutely impossible for him to look at things in the free, untrammeled way that the officers and board of direction of a bank are supposed to do.

(2) The fund in question is not the only resource possessed by the bank. There are dozens of others, including capital and surplus, and

* *Proceedings*, Am. Soc. C. E., for November, 1918.

if one fails or is in such form that it cannot be realized on at once, there are others that can be. In this way, the bank may be likened to an equalizing pond or reservoir for the benefit of all its depositors. The drain in one direction may be heavy, while that in others is light, and at the same time there is a constant supply from all sources. This condition could not possibly exist if the fund was directly invested by the owner. Mr. Gandolfo.

(3) The bank or trust company is in its turn a "Public Service Utility", subject to the most careful scrutiny by the State, and thus an additional safeguard is thrown about all sums placed here in trust. If the fund was privately invested by the owner, there would be no such safeguards.

Further, if the owner, as stated by the author, must always be ready to make good the depreciation and obsolescence, he in his turn must have funds immediately available with which to do so. As, following out this line of argument, he would ordinarily have all his personal funds invested, it would then be necessary for him to maintain some fund for this particular purpose. Wherein lies the difference, then, under such conditions, whether such a fund is held by the original party, or must go through several hands, except the far greater risk and uncertainty in regard to its ever materializing at its original source when needed.

As far as investing a sinking fund in extensions to the plant, the moment it is so invested, it becomes "Capital", subject at once in its turn to depreciation, and is no longer "Sinking Fund". Further, if the same rate of profit is allowed on the sinking fund as on the plant (and, under conditions as hereinafter described, there seems to be no question but that there should be), then there is no difference as to whether the fund is invested in new construction or not, as far as the returns to the owner are concerned.

As for a concrete example illustrating much of the foregoing, there is one in the New York, New Haven and Hartford Railroad. The funds of this road, which should have been used to take care of depreciation, obsolescence, and improvement, within the last few years have been used for other purposes, the details of which it is needless to go into here. When, a short time ago, on account of a series of accidents that cost the lives of scores of people, an investigation by the State disclosed the fact, among others, that sections of this road were still equipped with signals of a type which had been discarded as obsolete by other roads some fifteen or more years ago, and it therefore became imperative to use for improvements the funds that had been otherwise "invested", did the parties who had appropriated them make any move to replace them? When public opinion, however, at last aroused, demanded that something be done, $67 552 000 of 6% convertible debenture bonds were offered to the general public, in order to take care of some $46 023 750 of obligations which are to mature within com-

Mr. Gandolfo.

paratively short periods, the remainder being to pay for financing and to obtain funds to make good the depreciation and obsolescence. The issue of these bonds has been opposed by the present bondholders, and the offering of them is now awaiting the decision of the Court. In place of this, approximately $45 000 000 of short-term notes were issued, to pay for the immediately maturing obligations, and also to provide for pressing needs due to depreciation and obsolescence.

Further examples, such as the foregoing, might be given in detail, but there is very little use or profit to be gained by anybody in arguing theoretically about a question of this kind, when undeniable facts and occurrences are on record before the financial world and the general public.

As the author points out, at no time should a depreciation and obsolescence fund amount to a very large percentage of the cost of the plant, on account of the constant withdrawals from it to maintain the plant in first-class and up-to-date condition. If such a fund is actually in hand, that is, actually in trust, there is no question but that rates should be allowed on it, just the same as on any other item of the plant. If the plant is being kept in repair and up to date by constant withdrawals from the fund, there seems to be no reason that rates should not be allowed on the full invested capital of the plant. This matter can be treated in two ways, which lead to the same result: The property can be appraised at its full cost, without depreciation, and returns can be allowed on this figure, no special mention being made of the fund; or the property may be appraised at its cost, less depreciation, and returns may be allowed on this, and also on the depreciation fund. Of course, it is assumed that the sinking fund is always approximately equal to the actual depreciation at any time. A sinking fund, provided for depreciation and obsolescence, and actually in hand, is part of the plant, and should be treated as such.

The necessity of keeping a public utility property in first-class condition by constant withdrawals from the sinking fund can be further illustrated by the following theoretical case. Suppose that the life of the physical property has been determined, and then a depreciation fund is created which, at the end of the period, will just return 100% on the investment. At the end of this period, the plant, like the deacon's famous "one-hoss shay", would theoretically vanish into thin air, leaving the sinking fund in its place. If a fund under such conditions was allowed to accumulate, then the theory that a public service utility must render adequate and constant service to the public is untenable, and cannot be upheld, as the owners would have a perfect right to allow the plant to deteriorate gradually to nothing, or to wipe out the utility at such time as they saw fit, and leave the public without its benefits.

Mr. Gandolfo.

This question of constantly keeping up the depreciation of a property is referred to in Knoxville *v.* Knoxville Water Co., 212 U. S., 1, Jan. 4, 1909, which says that the Company "* * * is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years, the original investment remains as it was at the beginning". That public service corporations are expected to keep the plant up to date, without allowing any excessive accumulation of a sinking fund, is shown by the opinion of Judge Evans in Cumberland Telephone and Telegraph Co. *v.* City of Louisville, 187 Fed., 637, April 25, 1911, and that of Judge Haight in People *ex rel.* Manhattan Railway Company *v.* Woodbury, 203 N. Y., 231, Oct. 17, 1911.

On page 2055,* Mr. Alvord argues that if a sinking fund has been withdrawn from the plant and used for other purposes—or, as he puts it, for private gain—then the owners are not entitled to rates on it. The writer fails to see why it is necessary to argue on such a point as this, as it is a self-evident fact. If rates were to be allowed on something in the way of a sinking fund that does not exist, then it would be just as logical to remove an entire battery of boilers (or any other part of a plant), erect and use them in some other place, and then claim rates on their value, on the ground that the owners would replace them if needed. Thus the argument might be continued *ad infinitum* until the best part of a plant had been removed and rates were still being allowed on it as originally installed.

On page 2060,* under the 5th conclusion, the author says: "That though it is not now considered improper to use reserve or sinking funds allowed for depreciation for private gain * * *". The writer hardly thinks that Mr. Alvord can have realized what he was saying when he made this statement, or else that he did not mean what he says. If a private individual owns a plant himself, and sets aside a sinking fund for depreciation, and then after a time withdraws this fund and uses it for some other purpose of his own, but still calculates that he has a depreciation fund, he simply deceives himself—and there is no deception so fatal as self-deception. If, on the other hand, the concern is a stock company, and a sinking fund for depreciation and obsolescence has been created, and then the officers, directors, or managers should use this fund for their own private gain, it would amount to misrepresentation and fraud to both stock and bond holders on the one hand, and misappropriation and embezzlement of funds on the other, both of which are criminal offenses under our penal code. Furthermore, even if a board of direction did use such a fund in some other way, even if for the direct benefit of the stockholders, it is very doubtful if, under a strict code of ethics, it would have a moral right

* *Proceedings*, Am. Soc. C. E., for November, 1913.

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to do so, without first having obtained the majority vote of the stockholders to such a venture; although, in such a case, the strict letter of the law would doubtless uphold a board in such a proceeding.

In conclusion, it may be said, therefore, that a sinking fund to cover depreciation and obsolescence must be kept constantly in hand, and be available at all times; that constant withdrawals must be made from this fund to keep the plant in first-class and up-to-date condition (that is, 100% efficiency and value); that such a fund cannot be invested in other enterprises, as then it is no longer available for its purpose, and does not exist; and that, unless such a fund, or its equivalent in some form or other, exists, the plant is not entitled to full cost or replacement value, but must be appraised on a basis of cost or replacement value less depreciation.

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PAPERS AND DISCUSSIONS

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STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

By Messrs. A. C. Janni and L. J. Mensch.

Mr. Janni.

A. C. Janni, M. Am. Soc. C. E. (by letter).—Before entering into any discussion on the opportunity and legitimacy of the assumption on which this work hinges, the writer wishes to call the author's attention to a little mathematical "license" which he took in simplifying the expression, $P\,Q = \frac{v \sin.\alpha}{\cos.(\alpha + \beta)}$,† because it undermines, perhaps, the whole mathematical "make-up" of all the formulas which depend on that *"lapsus calami"*.

There is no doubt that, as α tends toward zero, $\sin.\alpha$ may be substituted for α; this simplification being entirely legitimate; but, if α tends toward zero, the expression $\cos.(\alpha + \beta)$ does not tend toward $\cos.\beta$, by any means.

The author knows that $\cos.(\alpha + \beta)$ is purely a symbol of a certain operation to be performed, and is not an actual value; its value is given by $\cos.\alpha \cos.\beta - \sin.\alpha \sin.\beta$, and it is on this latter that he may operate his simplification. Thus his $P\,Q = v\,\alpha \sec.\beta$ should become $P\,Q = \frac{v\,\alpha}{\cos.\beta - \alpha \sin.\beta}$, when it is assumed that α tends toward zero, to be a mathematically correct simplification.

Concerning the assumption adopted by the author in order to find a "workable formula" for the compressive stress in concrete, and

* This discussion (of the paper by William Cain, M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† See foot-note, *Proceedings*, Am. Soc. of C. E., for November, 1913, p. 2070.

Mr. Janni.

also some of his statements, the writer cannot quite agree. Two questions are before him, namely:

1. Do we need that formula?
2. Is that formula worked out correctly?

It is the writer's opinion that neither of these questions can be answered in the affirmative.

Theory is quite competent to take care of a design of a wedge-shaped beam, if necessary, despite what the author says. If the cross-section of a wedge-shaped concrete beam is rectangular and the angle, β, is not large, the common formula for bending is sufficiently good to give the maximum compressive stress in the concrete.

It is a fact that the formula entails the truth—which, by the way, is not a postulate—demonstrated by Saint-Venant, that all bending stresses at any point act perpendicularly to the cross-section; but this, in the case where β is not very large, does not prevent the formula from giving a result sufficiently near to the truth to insure the maximum stress.

That the direction of the maximum stress, in the case of Fig. 1, is parallel to the top, $N\ M$, of the beam, and on which the author lays so much stress, is, nowadays, mathematically ascertained.* In the case of a dam design, that direction is of capital importance, and the failure to recognize it, or the overlooking of it, in the design of some dams, has often been the cause of loss of thousands of innocent lives and, perhaps, millions of dollars. In a wedge-shaped beam, however (or in a counterfort), the conditions are such that the direction of the maximum compressive stress can be ignored in most cases.

If the cross-section of the wedge-shaped beam is not rectangular, then, with proper limitations, the usual formula to determine the principal stresses, reported in many textbooks, can be used. This formula is, precisely, designed for a beam by taking as unknown quantities certain inclined stresses, called "principal stresses", instead of the normal stresses in that section, and, really, this is the most logical way of designing a beam, although the common practice, for opportune reasons, follows that of the bending and shear formulas.

Winkler, in 1867, as far as the writer knows, wrote on this subject, and, after him, Culmann, Ritter, Guidi, and others treated the same question; so that the conception cannot appear novel, as the author seems to think, at least judging by some of his remarks.

If, finally, the inclination of $N\ M$, Fig. 1, is large, then Grashof's formula, with due limitations for the case of reinforced concrete, may be used.

In view of all the foregoing considerations, the writer thinks that there is no need of a formula for the case in point, and that, after all

* M. Levy, *Annales des Ponts et Chaussées*, 1897; and F. Platzmann, *Ueber den Querschnitt der Staumauern*, Leipzig, 1908.

the arbitrary assumptions of the author in reaching his "workable formula", admitting its correct mathematical "make-up", its results are much farther from the truth than those given by the existing formulas, which, in all cases, are very reliable. Mr. Janni.

It is necessary, accordingly, to take into consideration the author's assumption in finding that formula, and, as a consequence, its mathematical reliability.

As the writer said before, it is very well known that the extreme top compressive stress is parallel to the beam top as, disregarding the friction on the soil, Fig. 1, the maximum tensile stress is parallel to the bottom, *L I*.

The author states that the maximum tensile stress at that region of the beam would be parallel to *L I*, if the beam were a homogeneous one.

Even if the beam were homogeneous, if friction is not disregarded, the maximum tensile stress, at the bottom of the beam, is not parallel to the bottom face.

When the beam is heterogeneous, as in the author's case, equilibrium conditions do not change, if friction is included; the only difference being that, as usual, the tension in concrete is disregarded, so that the designer is inclined to disregard its direction, also, unless it is not a case of scientific research.

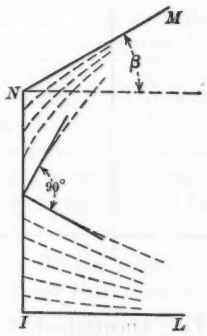

FIG. 9.

Now, the maximum compressive stress being parallel to the top, and the maximum tensile stress (always disregarding friction) being parallel to the bottom, the intermediate bending stresses, the author says, change their direction gradually from *N M* to *L I*, Fig. 1. As a matter of fact, those intermediate bending stresses do not change their direction gradually in the way the author means. Fig. 9 gives an idea of the behavior of the internal stresses in the cross-section, *N I*, on account of the angle, β. The author, however, states that, for the purpose of finding "workable formulas", he must assume that the intermediate compressive bending stresses be parallel to the maximum compressive bending stress. Such an assumption cannot be justified, when it is well known how those stresses act, and a designer knows how to determine them. Their direction, which is not constant for all, is a function of the angle, β, and not of cos.β, and their position is farther and farther from the author's assumption as β increases.

Apart from this assumption made by the author, which does not seem justifiable to the writer from any point of view, it is interesting to study the formula itself.

Mr. Janni.

Formula 8, which, according to the author, gives the maximum compressive stress in the concrete, will be taken into consideration. For the sake of clearness, the writer will compare the theoretical formula for bending in a beam of constant cross-section, which, when β is not very large, may be used without any fear of going astray, with the author's formula for the same kind of stresses in a wedge-shaped beam, using for both the same lettering, Fig. 10.

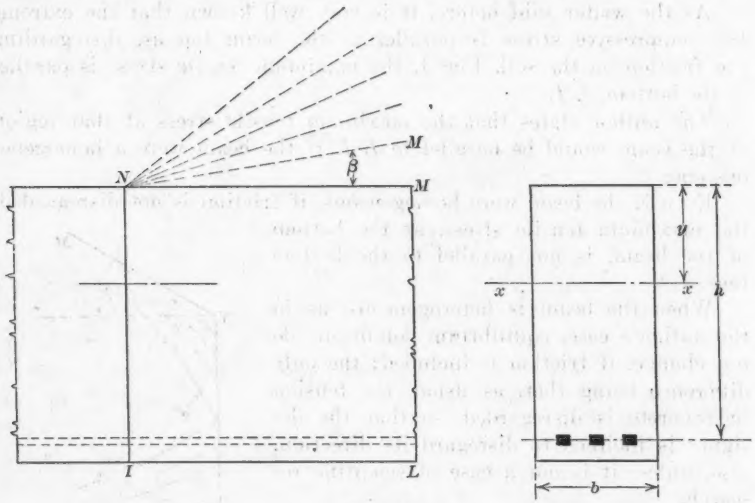

FIG. 10.

The theoretical formula for bending in a beam of constant rectangular cross-section is

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}y\right)} = Q \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

The author's formula for bending in a wedge-shaped beam is

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}y\right)\cos.^2\beta} = \frac{Q}{\cos.^2\beta} \quad \ldots\ldots\ldots\ldots (23)$$

It is plain, as the author says, that Formula 23, for $\beta = 0°$, becomes Formula 22.

Formula 23, however, as given by the writer, presents some other information rather interestingly.

Suppose that $N\,I$, Fig. 10, be a constant section, and that the line, $N\,M$, rotates about N, the angle, β, increasing gradually while M (external moment), keeps constant, the section, $N\,I$, according to

Formula 23, becomes weaker and weaker, until, when $\beta = 90°$, it has no resisting power whatever. Mr. Janni.

The writer does not think any comment necessary, except to recall the closing words of the paper: "The writer believes that he has effected a practical solution which is on the side of safety and may commend itself to the practitioner."

It is to be added, however, that Equation 8 is not the only equation by the author which leads to such conclusions; Equation 7, for instance, and its derivative equation for "rough computations," as the author designates it, are of a similar statical consistency.

Equation 3 has been worked out to enable the finding, according to the author, of the position of the neutral axis.

It is not clear to the writer, however, why the author gives a rather long equation (which, by the way, is not correct), when there are already several plain methods of finding that axis in a reinforced concrete beam.

The writer thinks it would be necessary for the author to devote a few words in justification of his point of view concerning Equation 3.

The writer, in closing, would like to ask the author the reason for his statement that "The unit stress, v, acts parallel to $N\ N'$". The writer has the impression that this statement, as put by the author, is not another of his assumptions, but a true and proper theorem concerning the equilibrium of internal forces in a wedge-shaped beam; and, as such, he would like further explanation.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—In the discussion of a paper on reinforced concrete construction before this Society several years ago, the statement was made that the counterforts of retaining walls do not act as beams. This alone would show that Professor Cain's paper is important. It also seems that the writers of textbooks have neglected to devolop theories of wedge-shaped beams, although quite a number of such books contain theories nearly identical with that of Professor Cain, for curved beams such as crane hooks, arched girders, etc. Mr. Mensch.

Professor Cain points out that an engineer is apt to overlook the fact that the compressive forces must be multiplied by $\cos.^2\beta$ when entered in the elastic equation, instead of by $\cos.\beta$ which one would be likely to do without going deeper into the matter. Probably a great many mistakes are made on this line, yet it is strange that they are made for the compression side of the beam, though on the tension side it is quite natural to multiply the actual cross-section (which is the $\cos.\beta$ value of the section made by the plane, $I\ N$) by $j\ d\ \cos.\beta$ to find the moment of the tensile forces.

For compression members, in trusses with inclined members, engineers are also accustomed to consider only the cross-section at right

Mr. Mensch.

angles to the direction of the stress; therefore, it appears to the writer that only carelessness or incompetence is the cause of the mistake complained of. Professor Cain's theory stands and falls with the fact that plane sections remain plane after deformation.

His assumption that the compressive forces are parallel to the compression face is certainly permissible in all cases where the percentage of reinforcement is low, say, less than 1%, and errs only slightly on the side of danger.

The law or assumption of plane sections remaining plane after deformation is certainly one of the greatest inventions of modern engineering, and enabled us to obtain a great number of simple working formulas, where formerly rules of thumb were used, yet the law is not universal, and is strictly proven only for straight beams of high-grade iron of rectangular section, within the elastic limit, and never for the neighborhood of ultimate load.

Professor Schule, of Zürich, about 12 years ago, proved that this assumption does not hold good for reinforced concrete beams, even under light loads, and why engineers all over the world have utterly disregarded these important tests is certainly strange.

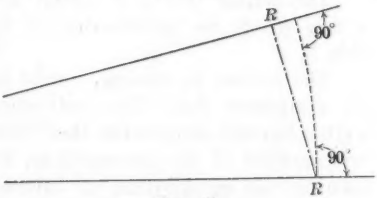

FIG. 11.

It is a well-known fact that the plane-section theory for plain concrete beams gives results which vary just 100% from actual tests, and still the same theory is applied to reinforced concrete beams. R. L. Humphrey, M. Am. Soc. C. E., in the discussion of his painstaking tests of 343 reinforced concrete beams, declares that the compressive stresses in the concrete and the tensile stresses in the reinforcement often differed 50% from the theory. In the face of such facts it may be expected that Professor Cain's table will differ just as much from facts, although it is much better than relying on a mere guess.

Tests of cast-iron beams prove that such wedge-shaped beams fail on a curved surface which is tangent to perpendicular lines to both faces of the beams, as shown in Fig. 11. For this reason the writer has advised* calculating such a counterfort for a section, $R\,R$, like a common beam, which greatly simplifies the computations and errs on the safe side. Engineers are accustomed to making a similar assumption in case of concrete arches, when they determine the stresses in a section at right angle to the center line of the arch. In view of this fact, it would be well to calculate the fillet in Fig. 4 along a line about bisecting the angle, $N'\,N\,I$, which will be found the dangerous section.

* "The Reinforced Concrete Pocket Book."

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.

Discussion.*

By Messrs. Allerton S. Cushman and Samuel Tobias Wagner.

Allerton S. Cushman, Assoc. Am. Soc. C. E. (by letter).—Mr. Sabin's exposition of the present situation in regard to paint protection of steel would appear to have at least the merit of simplicity. In a few brief paragraphs he sums up all that he cares to consider of the researches, theories, and conclusions of contemporary investigators of corrosion and paint problems, and sets forth what he terms the "whole history" of this "jargon" about inhibition and stimulation, which he states never had any particular value to the consumer, and is generally used to mislead him. Mr. Cushman.

It does not seem fair or just that such a statement should be allowed to pass unchallenged, in view of the great number of engineers and others who are responsible for the care of steel structures and believe in the value of the "inhibitive" quality of protective coatings.

Mr. Sabin's review entirely ignores the electrolytic theory of the mechanism of the reactions which lead to metallic corrosion, and brushes aside all its application to paint problems, as of little or no value. Ignorance, even in the purest sense of the word, of a mass of accumulated evidence and data, is not argument, and is likely to be considered on a par with King Canute's effort to order back the advancing tide. The electrolytic explanation of corrosion, on which the inhibitive theory of paint protection is based, is now accepted by a great number of investigators and authorities in America and abroad. As the writer, to the best of his knowledge and belief, is largely re-

*This discussion (of the paper by A. H. Sabin, Assoc. M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, and presented at the meeting of Jauuary 7th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Cushman.

sponsible for the "jargon" which Mr. Sabin claims has done so much to deceive consumers, he may perhaps be permitted to contribute a few paragraphs to Mr. Sabin's "whole history" of the subject under discussion. As far as the writer is aware, the first use of the words "inhibition" and "stimulation", as applied to corrosion and protection problems, occurred in his paper on "The Corrosion of Iron".* In that paper the electrolytic theory of corrosion was developed. A few excerpts from this paper will serve to illustrate the point made:

"It has long been known that rusting is inhibited and that polished iron will remain bright indefinitely in all sorts of alkaline solutions, provided they are sufficiently concentrated. This is also true of all solutions of salts of strong bases and weak acids which hydrolyze to an alkaline reaction. This fact has been eagerly seized upon by the adherents of the various theories which have been advanced, as it can be made to fit in more or less well with them all. Thus, alkalies absorb carbon dioxide, and therefore carbonic acid is prevented from carrying on its work of destruction. The added fact that fully saturated bicarbonate of soda also provides full protection to iron, even in fairly dilute solution, which would seem to be a stumbling block, has not shaken the faith of the devout believers in the carbonic acid theory.

* * * * * * * *

"Solutions of chromic acid and its soluble salts, such as the chromate and bichromate of potash, inhibit the rusting of iron immersed in them.

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"The writer has observed that if a rod or strip of bright iron or steel is immersed for a few hours in a strong (5 to 10 per cent.) solution of potassium bichromate, and is then removed and thoroughly washed, that a certain change has been produced on the surface of the metal. The surface may be thoroughly washed and wiped with a clean cloth without disturbing this new surface condition. No visible change has been effected, for the polished surfaces examined under the microscope appear to be untouched. If, however, the polished strips are immersed in water it will be found that rusting is inhibited. An ordinary untreated polished specimen of steel will show rusting in a few minutes, when immersed in the ordinary distilled water of the laboratory. Chromated specimens will stand immersion for varying lengths of time before rust appears. In some cases it is a matter of hours, in others of days or even weeks before the inhibiting effect is overcome.

* * * * * * * *

"If iron or steel is brought into contact with water, either pure or natural, iron goes into solution by replacing the hydrogen ions that are present, and if oxygen is present, the ferrous ions are oxidized with the formation of ferric hydroxide in the form of rust. Any substance which increases the concentration of the hydrogen ions in the water, such as carbonic, sulphuric, or other acids, will stimulate corrosion. On the other hand, any substance which decreases the concentration of the hydrogen ions, or in any manner prevents the interchange of

* *Proceedings*, Tenth Annual Meeting, Am. Soc. for Testing Materials, June, 1907, p. 211.

the electrostatic charge between the metal and the hydrogen ion, will inhibit corrosion. It follows from this that the rusting of iron is primarily a hydroxylization rather than an oxidation, and the oxygen does not directly attack the surface of the metal in the wet way or at ordinary temperatures. Mr. Cushman.

* * * * * * * *

"Substances which increase the concentration of hydrogen ions, such as acids and acid salts, stimulate corrosion, while substances which increase the concentration of hydroxyl ions inhibit it. Chromic acid and its salts inhibit corrosion by producing a polarizing or dampening effect which prevents the solution of iron and the separation of hydrogen."

These citations are sufficient to show the writer's responsibility in the application of the words "inhibition" and "stimulation" to the discussion of corrosion problems. In the same paper, the practical application of the use of chromic acid and its salts as an inhibiting agent was suggested and foreshadowed in the following words:

"From what has been shown in regard to the inhibitive action of the chromates it is not improbable, since such dilute solutions prevent electrolysis and corrosion, that the addition of small quantities of bichromate to boiler waters would be highly efficacious in preventing the rapid pitting which has caused so much trouble.

* * * * * * * *

"The experiment has been made of keeping iron and steel in dilute boiling solutions of bichromate, through which a current of air is bubbled, for protracted periods, and as long as the strength of the solution was equal to or above one or two pounds of bichromate to 1 000 gallons of water no rusting has ever taken place. There would, therefore, seem to be no reason why potassium bichromate should not come into use as a boiler protective. The application of the various inhibitors in the priming coats of paints and other protective coverings has already been to some extent made use of, and it would appear that slightly soluble chromates should be theoretically the best protectives for the first application to iron and steel surfaces."

At the 1908 meeting of the American Society for Testing Materials the writer presented a paper entitled "The Inhibitive Power of Certain Pigments on the Corrosion of Iron and Steel", and in that paper the whole subject of the inhibitive theory as applied to paint protection was developed further.

Following the presentation of the writer's 1907 paper, and in the discussion of a paper by Mr. L. S. Hughes, entitled "The Deleterious Ingredients in Paints", Mr. G. W. Thompson, who is quoted by Mr. Sabin, suggested the water pigment tests as described by Mr. Sabin, and described some experiments which he had already made on this subject. Mr. Thompson, in the course of his remarks, stated that the remarkable thing which appeared in his original experiments was that some of the pigments had inhibited the oxidation, one pigment having

Mr. Cushman.

caused a loss to the steel test piece of 0.37%, showing, according to Mr. Thompson's own words, a large inhibition, whereas another pigment had caused a loss of 2%, a considerable increase, as compared with the blanks which were run simultaneously. Mr. Thompson summed up by stating that he wished to make the suggestion to Mr. Cushman, Mr. Walker, and others who were investigating the corrosion of iron, that they make experiments to discover the relative electrolytic activity of various pigments in relation to iron. This suggestion was eagerly accepted by the writer, and, as Mr. Sabin describes, a committee of five chemists, from different parts of the United States, made a series of tests of the principal pigments on steel immersed in water. The results obtained by these laboratory investigations were then placed on the practical scale of experimentation by the steel test fence erected at Atlantic City by the Paint Manufacturers' Association of the United States, the inspection of which from time to time was placed under the supervision of suitable committees of the American Society for Testing Materials. As a result of these tests, which have been carried on constantly since 1908 and have been reported on annually, the best protection has been developed by the basic chromate of lead known as American vermilion. The pigments which received the five leading grade-marks in these tests depend on the chromate principle of inhibition. A number of the other leading pigments depend on the basic principle of inhibition, including both the orange mineral and red leads, which have also ranked comparatively high in the tests.

In spite of this, Mr. Sabin states:

"It is obvious that in a good paint the pigment particles are enveloped in a film of oil; they do not come in contact with the iron; if they did, the paint would peel off, for no dry pigment adheres well to metal. It is as true to-day as it has been in the past that steel rusts because air and moisture act on it; and paints are used to keep air and moisture from it. They do not inhibit rusting, except as they inhibit the cause of it."

This statement would appear to show that Mr. Sabin has not comprehended the manner in which inhibitive pigments tend to protect steel when mixed with oil and painted on the surface of the metal. No intimate contact of the pigment particle and the underlying surface is in fact necessarily called for, in order that the inhibitive action should be exerted. A simple experiment can be made to prove this, by any one interested in making it. If a slightly soluble chromate, such as zinc yellow or the precipitated chromate of calcium, is mixed and ground with linseed oil, painted on a steel surface, and allowed to harden in the air in the ordinary way, and if this painted steel specimen is then immersed in water, it will be found by the gradual yellowing of the water that a small part of the chromate pigment is capable of dissolving in the water and even of being leached out of

the film. This simple experiment shows that when a chrome inhibitive paint is used for protecting iron and steel, the water of the atmosphere which condenses or rains on a surface and is absorbed by the linoxyn coating, impregnates itself to a slight degree with the chrome radical of the pigment. This solution wets the metallic surface and to some extent maintains it in a passive or non-rusting condition. Though it is true that an ideal paint film should succeed in keeping air and moisture from the underlying surface, the fact is that paint films do not exhibit ideal qualities in this respect, and all of them are more or less permeable to water and atmospheric gases. Mr. Cushman.

It is well known that for many years red lead has been used more or less successfully as a prime coating material on iron and steel, and the writer has no criticism to make of properly manufactured red leads which are properly mixed with a good vehicle and properly applied to steel. Unquestionably, it must be admitted that these red lead prime coaters yield good protection. When, however, Mr. Sabin claims that nothing more can be done, and that there is no need of further investigation of paint protection problems, the writer must take issue with him, as it can easily be shown under test that, based on the inhibitive explanation of protection, many pigments are capable of giving better protection than even the best red lead. As a matter of fact, if the use of red lead as a prime coating material has already solved the problem of protecting steel to the maximum degree, it may well be questioned why it is that so large a percentage of structural steel all over the world is suffering from rapid corrosion and deterioration. Most of the inhibitive prime coating materials must inevitably cost more than red lead, and where, as is generally the case, the first cost of application is a factor of the highest importance, red lead will probably continue to be generally used. It is the writer's belief, however, that where the first cost of application is considered in relation to the maintenance or upkeep cost of structural steel for a number of years, it is now possible, as the result of research and investigation, to design an inhibitive prime coating material which will protect steel more efficiently than the usual red lead application, and he sincerely hopes that engineers and other consumers will not remain satisfied that the last word has been said with respect to paint protection in Mr. Sabin's presentation of the present situation in regard to the painting of structural steel.

Samuel Tobias Wagner, M. Am. Soc. C. E. (by letter).—The question of the selection of a proper paint to be used on a steel structure is one of the greatest importance, and much progress has been made in the last generation toward obtaining purer materials and applying them in a more workmanlike manner. For a number of years the writer has been of the opinion that it was not fair to expect any single Mr. Wagner.

Mr. Wagner.

paint to give the best results under different conditions of climate and exposure, and that generally specific conditions could be ascertained to determine which kind should be used. Throughout this discussion there were two general features which were kept prominently in the foreground; First: the materials should have a high degree of purity; Second, they should be very finely ground.

For the past 20 years the writer has looked on red lead in linseed oil as one of the best primers for structural steel, and, when mixed with some other equally pure materials, as the best final coats. The first large contract under which he used this material was in 1897 on some bridges, aggregating about 5 000 tons, on which the following specification was used:

"Red lead in oil shall be of a good bright color and very finely ground. The pigment shall contain at least 95% of red oxide of lead; no sample will be accepted that contains more than 2% of foreign matter that is vitrified, or that contains metallic lead; or that when mixed with linseed oil and drying japan without grinding, and applied in a good body to a vertical surface of iron will not dry without running or separating."

The steel on this contract was erected promptly after fabrication, and no special opportunity was given to examine the wearing qualities of the primer, but it was undoubtedly the fact that there was difficulty in filling the specification, and, when filled, the resultant paint was of such a character that it could not be applied smoothly, but either ran or was streaky. It always had to be freshly mixed in order to work it at all. The general results, however, were good.

Within the past 5 years the following experience was obtained with about 27 000 tons of structural steel bridges divided into several contracts. Nearly all the work on each structure was fabricated at one time, and the conditions on the work were that only half could be erected at that time, the other half having to be stored. Linseed oil was the primer specified on the first lot of contracts and after storing over a single winter it was found necessary to clean and re-oil a very considerable quantity of the metal. The specifications on the remaining contracts were modified by requiring that the priming coat should consist of red lead of the same specification as given above. Experience showed that after storage of more than 2 years the metal was in good condition.

The red lead required at this time was not obtained without very considerable trouble. The samples first submitted to test showed 49.08 and 56.15% of red lead; they were rejected, and the paint-maker promised to do better. Three samples were then submitted, showing 77.66, 79.56, and 83.26%, respectively, and these were rejected. These rejections caused a storm of protest, and it was stated that it was impossible to obtain a higher percentage of red lead in a mixed paint. After

Mr. Wagner.

waiting some time the general contractor changed his paint man, and the next two samples showed 96.48 and 96.68%, respectively. The next three samples showed 98.11, 98.41, and 98.63%, and it is possible that if more paint had been needed the figures would have been higher. The paint when applied to the steel had a good bright color, and had none of the streaky appearance of the red lead of 1897. The color of such a paint is noticeably different from a paint with 75% of red lead. The fact that such paint with 94% and more of true red lead, as prepared by recent methods, will not set has not been noticed by the writer, but this has been probably because he has not followed up all the details. It is an important matter, and the manufacturers are to be congratulated on effecting this improvement, as well as the fact that it has greater covering value.

The writer was one of the railroad engineers who was requested by the Committee of the American Society for Testing Materials to examine the paints applied on the Pennsylvania Railroad Bridge over the Susquehanna River at Havre de Grace, and can testify as to the good appearance of the paints which contained red lead in the finishing coats. It is also of common knowledge that in many cases, when all paint has disappeared from structural steel, it is possible still to distinguish the shop marks of red lead where that paint has been used.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILHELM HILDENBRAND, M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1908.

Wilhelm Hildenbrand, one of the most eminent designers and constructors of suspension bridges, was born on June 1st, 1843, at Karlsruhe, in the Grand Duchy of Baden, Germany. After receiving a classical education in the Lyceum of his native town, he entered its Polytechnic School, which was then the most renowned in Germany, having among its professors, Redtenbacher, one of the founders of scientific mechanical engineering, and Sternberg, equally prominent in scientific bridge design. After he was graduated in engineering and had passed the "Staats Examen" required for Government employment, he entered its service as Engineer of Highway Construction and as Inspector of rails at the rolling mills of Westphalia and Rhenish Prussia. After a year in the service of the State, he emigrated in 1867 to America.

His first position was as Draftsman in an architect's office, but he soon found employment with the late John A. Roebling, M. Am. Soc. C. E., who was at that time planning the construction of the Brooklyn Bridge. Under Mr. Roebling's directions, Mr. Hildenbrand made the first drawings of that famous structure, in the building of which he afterward took a prominent part.

During the delay caused by the initial difficulties of this enterprise he entered, as Assistant Engineer, the service of the New York Central Railroad where he was entrusted with the architectural design for the Forty-second Street Terminal Passenger Station in New York City, and with the construction of the great arched roof over the train-shed. Both structures were built in accordance with his plans and specifications, thus laying the foundation of his reputation as an architect and engineer. The roof truss had the largest span in the world at that time, and lasted until the entire reconstruction of the station required its removal.

In 1870, when the Grand Central Station was nearly completed, the actual construction of the Brooklyn Bridge began, and Mr. Hildenbrand was engaged by Washington A. Roebling, the Chief Engineer, as Principal Assistant Engineer. During Mr. Roebling's protracted illness, which confined him to his bed or room for nearly ten years, Mr. Hildenbrand made all the scientific investigations and mathematical

* Memoir prepared by Joseph Mayer, M. Am. Soc. C. E.

calculations necessary for the structure. He also made the architectural design for the approaches and had charge of the steel superstructure, which was designed, inspected, and erected under his direction.

In 1883, after the completion of the Brooklyn Bridge, Mr. Hildenbrand opened an office in New York as Consulting Engineer and, as Chief Engineer, built many suspension bridges in the United States and Mexico. He also built a truss bridge over the Ohio River at Wheeling, W. Va., which, like most of his work, is distinguished for its pleasing appearance.

In 1885 he submitted a design in the public competition for the Washington Bridge, and was awarded the second prize. Several features of his design were incorporated in this bridge as built.

In 1894 Mr. Hildenbrand did important work for the New York Chamber of Commerce which protested against the building of a pier near the center of the North River for a railroad bridge at that time projected by the New York and New Jersey Bridge Company. The President of the United States appointed a commission of five distinguished engineers to examine the feasibility of a single-span bridge across the river. Mr. Hildenbrand submitted a design, an estimate, and an argument in favor of a suspension bridge, and appeared before the Commission as the representative of the Chamber of Commerce. The Commission reported in favor of the feasibility of a suspension and against a cantilever bridge, which was the question at issue; Mr. Hildenbrand, therefore, carried his point.

In 1895 he became Chief Engineer of the reconstruction of the Covington and Cincinnati Bridge over the Ohio River. This was his most important and difficult independent work. The bridge is the third largest suspension bridge in the world, and was built originally by Mr. John A. Roebling in 1867, but had become inadequate for the increased traffic. Mr. Hildenbrand replaced the floor and the stiffening trusses with a wider floor and new stiffening trusses without interrupting traffic, and supplemented the old cables and anchorages by additional ones. The task of distributing the load between the new and the old cables was difficult, and required accurate calculation and delicate adjustment. It was evidently accomplished successfully; the new bridge is not only an adequate but a beautiful structure, and an ornament to the two cities. It was finished in 1899, to the satisfaction of his clients.

On the completion of this bridge, Mr. Hildenbrand went to Mexico, where he constructed a light suspension bridge, of about the same span as the Cincinnati and Covington Bridge, for the transport of silver ore from the mines of the Peñoles Company at Mapimi to the terminal point of a rack railway, built on the Abt system, which Mr. Hildenbrand had constructed in 1897.

As the representative of the Abt system of inclined rack railways, Mr. Hildenbrand, in 1889-1890, constructed the track of the Pike's

Peak Railway in Colorado, reaching an elevation of 14 214 ft., and furnished its locomotives.

In 1900, in the service of John A. Roebling's Sons Company, of Trenton, N. J., he became the Chief Engineer of the contractor for building the cables of the Williamsburgh Bridge. These cables, 50% larger than those of the Brooklyn Bridge, were built by him in one-third of the time required to construct those of the Brooklyn Bridge.

In 1904, he was appointed Consulting Engineer of the Westinghouse Electric Company, of Pittsburgh, and as such he designed the overhead structures for the electrification of the passenger traffic of the New York, New Haven and Hartford Railway, near New York City. He designed other and similar structures for the Westinghouse Company, acting as Consulting Engineer for this Company until his death.

In 1877 Mr. Hildenbrand published a treatise on "The Theory and Construction of Wire Cables," which is a standard work on the subject. In 1888 he published a book on "Underground Haulage of Coal by Wire Rope," which was awarded a medal and diploma of honor at the World's Fair of Chicago in 1893.

In 1895 he published in the German language a lecture on the history of suspension bridges from the earliest to the latest times. He also published many articles in the technical press, mostly referring to suspension bridges.

As an engineer and architect, Mr. Hildenbrand always showed good taste, and gave close and painstaking attention to all details. His extreme conscientiousness, which made him hesitate to delegate minor parts of his work to subordinates, was the only obstacle to his attaining a still more brilliant success than he achieved.

He had a very interesting and original personality, and was a true friend to many who were fortunate enough to make his acquaintance. He was liberal to a fault in his assistance to many who needed it. He was an enthusiastic admirer of Wagner, played his operas on the flute, and never missed attending their performance when opportunity offered.

He was married, in 1901, to Miss Hubbard, daughter of Judge Hubbard of Covington, Ky., who survives him.

Mr. Hildenbrand was elected a Member of the American Society of Civil Engineers on February 5th, 1902.

GEORGE BROWNE POST, M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1913.

George Browne Post, the son of Joel B. and Abby M. Post, was born in New York City, on December 15th, 1837. He was educated at Churchill's Military School at Sing Sing, N. Y., and the Scientific

* Memoir prepared by the Secretary from information supplied by George B. Post and Sons.

School of New York University, from which he was graduated as Civil Engineer with the Class of 1858.

After his graduation, and during 1858, 1859, and 1860, he studied Architecture with the late Richard M. Hunt. In the latter year he formed a partnership with a fellow-student, Mr. Charles D. Gambrill, for the practice of Architecture.

During the Civil War, Mr. Post served in the Army for 4 months in 1862 and for 4 months in 1863, as Captain in the Twenty-second Regiment, New York Volunteers. At the first battle of Fredericksburg, he acted as Volunteer Aide on the staff of General Burnside, commanding the Army of the Potomac. Mr. Post was promoted to the rank of Major, Lieutenant-Colonel, and then Colonel of the Twenty-second Regiment.

After the War he resumed his professional career, and his partnership with Mr. Gambrill was dissolved. In 1905, with his sons, William S. and J. Otis Post, he formed the firm of George B. Post and Sons.

Among the buildings designed by Mr. Post during his long career, are the Prudential Life Insurance and the Mutual Benefit Life Insurance Buildings in Newark, N. J.; the Wisconsin State Capitol; the Cleveland Trust Company Building, in Cleveland, Ohio; and the Manufacturers' Liberal Arts Buildings at the Chicago Exposition in 1893. In New York City, he designed the buildings for the College of the City of New York; the New York Produce Exchange; the New Stock Exchange; the Pulitzer Building, and the Western Union Building, in Dey Street. He also designed the Equitable Life Assurance Society Building, and several well-known private residences.

Mr. Post was a Member of the Architectural League of New York, and served as President from 1893 to 1897, inclusive. He was elected an Honorary Life Member in 1912. He was a Fellow of the American Institute of Architects, of which body he was President from 1896 to 1899, inclusive. He was a member of the New York Chapter of the American Institute of Architects, and President in 1904; of the Fine Arts Federation of New York, and President in 1898. He was a Charter Member of the National Arts Club, and President from 1898 to 1905. He was a member of the Municipal Art Society, and served as Director from 1901 to 1909. He was a member of the Council of the National Sculpture Society in 1904. He was also a Member of the National Institute of Arts and Letters, the American Academy of Arts and Letters, the Province of Quebec Association of Architects, New York Academy of Sciences, American Geographical Society, National Society of Craftsmen, Public Art League, Archæological Society of America, National Geographical Society, and the Metropolitan Museum of Art. In 1907, he was appointed Honorary Corresponding Member of the Royal Institute of British Architects. He

was elected an Associate of the National Academy of Design in 1907 and an Academician in 1908.

In 1901, Mr. Post was decorated a Chevalier de la Legion d'Honneur of France. In 1908 he received the honorary degree of Doctor of Laws from Columbia University, and in 1910, he was awarded the Gold Medal of the American Institute of Architects.

When the Tenement House Commission, known as the Gilder Commission, was appointed by the New York State Legislature, he was made a member, and, in 1902, he was appointed a member of the Board of Commissions of the St. Louis Exposition by the Governor of New York.

Mr. Post was appointed, by the Secretary of State, as a delegate to represent American Architects at large at the World's Congress of Architects held in London, and, in 1906, the Secretary of Agriculture appointed him a collaborator of the Forest Service of the United States Department of Agriculture. In 1906, President Roosevelt made him a member of the National Advisory Board on Fuels and Structural Materials, to which Board he was reappointed in 1907, 1908, and 1909. In 1909, he was appointed a member of the Bureau of Fine Arts by President Roosevelt.

He was appointed a member of the Committee of Patronage to the Eighth International Congress of Architects in 1907, and in 1908 a Member of the Permanent Committee.

He was a member of the Expert Committee to appoint a sculptor and select a design for the Lafayette Monument erected in the courtyard of the Louvre in Paris.

In 1863, Mr. Post was married to Miss Alice M. Stone, daughter of William W. Stone.

He was a member of the New York Chamber of Commerce, the New Jersey State Chamber of Commerce, Century Association, Union Club, Cosmos Club of Washington, D. C., Lawyers' Club (Charter Member), and the New York Farmers Club.

Mr. Post was elected a Member of the American Society of Civil Engineers on September 2d, 1896.

CHARLES WALKER RAYMOND, M. Am. Soc. C. E.*

DIED MAY 3D, 1913.

Charles Walker Raymond was born at Hartford, Conn., on January 4th, 1842, and was the second son of the Reverend Robert R. and Mary A. (Pratt) Raymond. During his childhood his parents removed to Syracuse, N. Y., where he was edu-

*Memoir prepared by Dr. R. W. Raymond and Alfred Noble, Past-President, Am. Soc. C. E.

cated in the public schools and High School until 1857 when, his father becoming Professor of the English Language and Literature in the newly established Collegiate and Polytechnic Institute of Brooklyn, N. Y., the family moved to that city, and he entered that institution. As a child, he had been intensely interested in literature, but backward and awkward in mathematics. In the Polytechnic he came under the influence of Professor Richard Smith, a retired officer of the Engineer Corps (afterward President of Girard College), who was Professor of Mathematics; and, to the surprise of his family, he began to distinguish himself in that study. The strangest feature of this change was that he developed, under the inspiring touch of his brilliant teacher, not a previously unsuspected genius for mathematics, but a high ambition to conquer by persistent work the field which seemed so difficult. It was by hard work at every step that he acquired a thorough knowledge which placed him at the head of his class, and led Professor Smith to urge that he should seek an appointment as cadet in the United States Military Academy, on account of his unusual mathematical ability. In these days, when it is the fashion to educate boys mainly in the lines to which they show the earliest bent, this instance of training in directions not congenial is suggestive. We hear much about the subsequent failures of those who took the highest rank in school; but when that distinction is won, not through the easy superiority of genius, but by intense devotion to distasteful work, the man who wins it is likely to keep it always. This, at least, was the keynote of General Raymond's career, and the secret of his remarkable record of unbroken success through more than forty years of professional service.

Graduated from the Polytechnic in 1860, he was appointed in 1861 to the Military Academy, where he stood at the head of his class for four years. On his graduation, in 1865, he was immediately assigned to the Corps of Engineers with the full rank of First Lieutenant—skipping the rank of Second Lieutenant altogether. In his "furlough year" (1863), he had spent his leave of absence as an officer on the staff of Major-General Couch, by special appointment of the Secretary of War. General Couch commanded the Department of the Susquehanna, and Lieutenant Raymond's service with him in June and July covered the Campaign and Battle of Gettysburg.

From October 1st, 1865, to September 17th, 1866, he was Assistant to the Special Board of Engineers for improving the fortifications near Boston, Mass. He was then transferred to the Pacific Coast, where he served for a brief period as Assistant Engineer in the construction of the defences of Alcatraz Island, San Francisco Harbor, and then became Recorder of the Board of Engineers for the Pacific Coast. This position be held from December 28th, 1866, to March 3d, 1869, during which period he was promoted (March 1st, 1867) to be Captain

in the Corps of Engineers, and also conducted important operations at Lime Point, San Francisco Harbor, where he was Assistant Engineer for two periods—April 24th to July 7th, 1868, and November 5th, 1868, to March 3d, 1869. The work at Lime Point involved the removal of large masses of rock by heavy blasts. The interval between the two periods named was occupied with the reconstruction of Fort Stevens, at the mouth of the Columbia River, Oregon.

In 1869, after the purchase of Alaska by the United States, the American Fur Company, which had succeeded to the business of the Russian Fur Company, sent a small party up the Yukon River as far as Fort Yukon, a post of the Hudson Bay Company, to investigate the fur trade of the upper river and decide whether to establish an American station there. Captain Raymond was ordered by the Commanding General of the Department of the Pacific to accompany this party, for the purpose of making a reconnoissance of the lower Yukon, and determining astronomically the position of the meridian which constitutes the Eastern Boundary of that part of Alaska. Accompanied by one assistant and furnished with an astronomical field outfit, he joined the traders' party, which left San Francisco on April 6th, 1869, in a sailing brig, carrying on deck a small steam pinnace, and bound for Sitka, at which point his party was reinforced by one private soldier detailed for this service. Leaving Sitka on May 9th, the expedition, after many delays, reached the mouth of the Yukon, launched the little steamer, and proceeded in it up the river to Fort Yukon, a distance of, perhaps, 1,000 miles. During this trip, Captain Raymond and his assistant, relieving each other day and night, made a reconnoissance from which he afterward prepared the map of the lower Yukon which accompanied his official report and which was undoubtedly the best which had appeared up to that time. Arriving at Fort Yukon on July 31st, 1869, the traders' party soon ascertained that the fur trade of that region was not large enough to warrant the establishment of an American agency; and, as the freezing of the northern tributaries of the river was already threatening to make it too shallow for their steamer, they resolved to return at once, leaving Captain Raymond and his two companions to finish the astronomical work and make their way to the coast by themselves, but promising to hold the brig for them at the mouth of the river until a certain date. Establishing a field observatory, and observing the solar eclipse of August 7th, Captain Raymond determined the meridian boundary, proved that the British post was on American territory, and raised the American flag over it. (The post was immediately removed to the other side of the meridian.)

Leaving Fort Yukon on August 28th, his little party descended the river in a rude boat constructed by themselves—the Indians refusing to venture the voyage in their canoes at that season. This vessel con-

veyed them precariously as far as Anvik (about 500 miles from Fort Yukon and 500 miles from the sea), where it went to pieces. The remainder of the journey was characterized by hardship, exposure, and even starvation; and the explorers, exhausted and emaciated, reached the appointed rendezvous just as the brig was hoisting sail for San Francisco.*

From January 7th, 1870, to June 16th, 1871, he was Secretary of the Board of Engineers for Fortifications, etc., etc., of the United States; and from the latter date to August 23d, 1872, he commanded the Engineer Company at Willets Point, N. Y. On August 28th, 1872, he became Principal Assistant Professor of Natural and Experimental Philosophy at the U. S. Military Academy, and filled this position until February 27th, 1874, and again from August 31st, 1875, to July 1st, 1878—the interval being spent on special service in command of the U. S. Expedition to Northern Tasmania to observe the transit of Venus. From August 28th, 1878, to August 27th, 1881, he was Instructor in Practical Military Engineering, Signaling and Telegraphy, and during a part of that period served also as Superintending Engineer of Construction of the West Point Water-works, the Cadet Hospital, and the Cadet Barracks Extension. In the various positions mentioned, Captain Raymond spent more than seven years in actual service at the Academy.

During this period, he published an essay on Terrestrial Magnetism, which added to his already established reputation as a mathematician and physicist.

From August 30th, 1881, to January 13th, 1883, he commanded the Engineer Company at Willets Point. In January, 1883, he was placed in charge of river and harbor improvements, surveys, and coast defences in Massachusetts, and retained this position until February, 1886. The following condensed catalogue of his duties, compiled from the Annual Reports of the Chief of Engineers, will give some notion of their extent and variety: The protection and improvement of Boston Harbor, in which he established a practicable 28-ft. channel to the ocean; operations of similar character at the Harbors of Newburyport, Scituate, Plymouth, Provincetown, Lynn, etc., and Sandy Bay; studies and improvements of the Merrimac and Malden Rivers, and numerous other preliminary examinations, surveys, and projects. The most important of these works, from an engineering standpoint, was the improvement of Newburyport Harbor, at the mouth of the Merrimac, where Captain Raymond, through modifications of plan and method, secured, not only an increase of channel depth on the bar from 7 to 18.5 ft., but a saving in expense over the original scheme,

* Captain Raymond's official report, published as Senate Executive Document No. 12, of the Forty-second Congress, says little about the sufferings of his party, but they can be read between the lines. His health was permanently impaired.

which contemplated only a 17-ft. channel. His report on Sandy Bar (1884) contains a brilliant and novel discussion of the cross-section of a breakwater, and the anchorage capacity of a harbor, which contributed new ideas to engineering literature. On February 20th, 1883, he was promoted to be Major in the Corps of Engineers.

During his term at Boston, he served also for the greater part of 1883 and 1885 as Engineer of the First and Second Lighthouse Districts (covering the Coast of Massachusetts and Maine); and from October, 1883, to April, 1884, he superintended the removal of a wreck from Gloucester Harbor.

In February, 1886, he was ordered to take charge of levees and other improvements on the Mississippi River, from Warrenton to its delta, and from that position he was called, December 7th, 1886, to Washington, D. C., as Assistant to the Chief of Engineers. This office (which often made him temporarily Acting Chief of the Corps), he retained until January 26th, 1888, when he was appointed Engineer Commissioner of the District of Columbia—one of the three Commissioners constituting the government of the District. He served until February 1st, 1890, dealing, among other works of municipal engineering, with the difficult problem of electrical subways, concerning which his official report furnished a valuable theoretical and practical discussion.

From February 13th, 1890, to September 30th, 1902, he was (with the exception of two months in the latter year) continuously in charge of the defenses, harbor improvements, etc., at Philadelphia and in Delaware River and Bay. This work included the completion, on a novel plan and by a method designed by Major Raymond, and with great saving in both time and cost, of the now famous Delaware Breakwater which, disproving all the sinister prophecies of its early critics, stands unmoved after ten years of practical trial, a monument to his courage and originality as an engineer.*

The following sketch of his work in the Philadelphia District is compiled from the Annual Reports of the Chief of Engineers, from 1890 to 1902, inclusive.

For the first eight years of his tour of duty, the project for the Delaware River up to Philadelphia provided for a depth of 26 ft. at mean low water, and from Philadelphia up to Trenton for a depth of 12 ft. In the upper river above Philadelphia the principal obstruction to the 12-ft. channel was Kinkora Bar. At this point a dike had been built before he took charge. The natural depth was 7.5 ft. He suc-

* This work is described in the "Final Report of Lt.-Col. Charles W. Raymond, Corps of Engineers, Upon the Improvement of Delaware Breakwater," Reports of the Chief of Engineers, U. S. Army, 1899, Part II, p. 1346 *ff*. It is a significant circumstance that when Major Raymond took charge of it, in 1891, a plan had been already approved, in 1890, by the U. S. Board of Engineers, which he was expected to carry out; so that, in order to put his own conception into effect, he had to move that Board to recede from its own recently adopted scheme and adopt a new one, comprising features not supported by engineering precedents.

ceeded in obtaining greater depths, but the work done was not permanent in effect.

During these eight years he made annual examinations of the shoals in the river at Smith's Island Bar, Mifflin Bar, Schooner Ledge. Five Mile Bar, Bulkhead Shoal, Cherry Island Flats, Dan Baker Shoal, and other places. At most of the shoals and bars he obtained the projected depth of 26 ft.

Five Mile Bar is just above Philadelphia. It was improved by a dike and by dredging. Smith's Island was completely removed by dredging. Petty Island was partly removed. Ledges of rock were removed by blasting and dredging. Mifflin Bar was improved by dike construction and dredging. Schooner Ledge was deepened by rock removal. Cherry Island Flats was deepened by dredging. Bulkhead Shoal was deepened by the construction of Finn's Point dike. Dan Baker Shoal was originally planned to be deepened by a long dike from Reedy Island to Liston's Point, but this plan was later abandoned by reason of a decrease in the cost of dredging, and the channel was deepened by dredging alone.

In 1898 the condition of the river was good except at a few points. At Dan Baker Shoal the least depth was 16.5 ft. at low tide. In that year, a survey was made for the purpose of determining the possibility of still greater improvement, and on the basis of this survey a Board of Officers recommended a project for a channel from Philadelphia to Delaware Bay, 30 ft. deep and 600 ft. wide. Major Raymond started this improvement, deepening the channel to 30 ft. as rapidly as funds were made available. As the Delaware River is a silt-bearing stream which tends to obliterate artificial channels not placed exactly where Nature would place them, the improvement involves continual maintenance work. Even ten years after he had left the district, the work was still in progress, and it will always be continued as long as the river is used for commerce.

One of the great problems involved in this undertaking consisted in finding suitable places to dispose of dredged material. Major Raymond combined a dike for the improvement of Dan Baker Shoal with a plan for a disposal basin, making in mid-stream an artificial island about three miles long. The bulkhead was built, and the basin was so large that it has received dredged material for many years and is not yet full. This island is locally known as Raymond Island.*

*The following minute of the Executive Council of the Philadelphia Board of Trade shows the estimate placed on his work by that body:

"To the President and Members of the Executive Council of the Board of Trade:

"GENTLEMEN:

"Your Committee on the Improvement of the Harbor and Delaware and Schuylkill Rivers respectfully reports:

"That it has noted with regret the death of General C. W. Raymond, U. S. A., and in view of the services rendered by him to the city of Philadelphia during the twelve years he was in charge of this district as Engineer Officer, presents for your adoption the following minute:

"The Executive Council of the Philadelphia Board of Trade records this minute

In May, 1898, he became Lieutenant-Colonel, Corps of Engineers, and in May, 1901, a member of the U. S. Board of Engineers, of which, as a young Captain, he had been the Secretary, 31 years before. He remained a member of this Board until his retirement from active service in 1904.

During this period he completed the great work of an analytical index of the Reports of the Chief of Engineers from 1866 to 1900, inclusive. This book, in three volumes, is a model of intelligent and convenient classification, as well as comprehensiveness and accuracy.

It was in these later years of his life that some of his most important work was performed. As already observed, he served as member on many Boards and Commissions, reporting on fortifications, rivers, harbors of refuge, etc. Among these, the most noteworthy were: The Board to decide between San Pedro and Santa Monica Bay, for the location of a deep-water harbor on the coast of California; the Board to determine the maximum span for a suspension bridge, and especially for such a bridge over the Hudson River at New York; and the Board to determine the route and cost of a deep waterway from the Great Lakes to the Atlantic.

Of the two latter, he was the President. The report of the Bridge Board contained an analytical discussion of the Theory of Suspension Bridges, mainly prepared by him, a translation of which was subsequently used in Europe as a textbook of instruction. The Deep Waterways Board made careful surveys for a ship canal from Lake Erie to deep water on the Hudson River, below Albany, and, with the aid of maps furnished by the Engineer Corps of the Army for the chan-

upon the death of General C. W. Raymond, U. S. A., which took place after a lingering illness May 3, 1913.

"The city of Philadelphia owes a debt of gratitude to General Raymond for his intelligent and untiring efforts for the improvement of the navigation of the Delaware and Schuylkill Rivers and the removal of Smith's and Windmill Islands, which made possible a widened Delaware Avenue, and the building of wharves of such dimensions as to accommodate modern vessels.

"The work on the improvement of the harbor was prosecuted under his direction from the time the title to the islands was vested in the United States, under condemnation proceedings, May 29, 1890, until the completion of the readjustment of the harbor conditions, January 10, 1898, and during all that time he was responsive to every suggestion of the commercial and maritime interests looking to the early and successful termination of the work which has inured so greatly to the advantage of the city and port.

"It was under General Raymond (then Major) that the initial dredging for a 30-ft. channel, as provided by Act of Congress, March 3, 1899, took place, and was continued until he was relieved from duty at Philadelphia on July 20, 1901. He designed and practically carried to completion the great constructions belonging to the National Harbor of Refuge at the entrance of Delaware Bay.

"General Raymond's pre-eminent qualifications as an engineer were universally acknowledged, and his appointment as Chairman of the Commission to plan and construct the tunnel approaches to the New York Terminal of the Pennsylvania R. R. Co., was a practical recognition of his high standing in his profession; and the success of the undertaking furnished additional proof of his unrivaled technical skill and executive ability.

"The members of the Executive Council in adopting this minute, desire to express their high appreciation of the services of incalculable value rendered the port of Philadelphia during the twelve years General Raymond had charge of this district, and at the same time to tender his family their sympathy in the loss sustained by them.

"On motion, this minute was adopted by a rising vote and the Secretary instructed to send a certified copy to the family of General Raymond."

nels above Lake Erie, prepared detailed estimates for the whole line from the head of lake navigation. The report of the Board, which was the joint work of its members and assistants, was universally recognized as the most complete and thorough of its kind in the literature of engineering.

In January, 1904, he became Colonel, Corps of Engineers; and on June 11th of the same year, he was retired, at his own request, with the rank of Brigadier-General, U. S. A., after more than forty years of consecutive active service. The Government, however, still required him as one of its representatives in the International Congress of Internal Waterways, of which he was, from 1902 until his death in 1913, a member of the Council, Chairman of the American Section, an attendant at the meetings of the Congress, and a contributor to its Proceedings. The latest of these meetings was held in 1912 in the United States, and General Raymond would have been its presiding officer, had he been able to be present.

One of the most memorable labors of his life was the last. Already several years before his retirement, he had been permitted by the War Department, at the urgent request of the Pennsylvania Railroad Company, to act as Chairman of the Board of Engineers created by that Company to supervise the design and construction of the vast improvement contemplated by it in and around New York City, including the tunnels under the Hudson River, the East River, and the Borough of Manhattan; the great Pennsylvania Terminal in New York; and the terminals and yards on Long Island and in New Jersey. General Raymond was not an idle member of that supreme body; all important features of the plans and specifications were passed on by the Board, to the work of which he gave unremitting attention for more than eight years. The many studies and investigations carried out by him during that period were fruitful in determining the final design. Among the most important results of his special labors was the discovery of a minute diurnal rise and fall of the tunnel tube, under the influence of the tides. His recommendation that the tunnel should be left free to move, without any attachment of piles to resist either rise or fall, was adopted by the management of the Company, and has been vindicated thus far by experience. The great tube now adjusts itself freely to the changes in pressure of the material surrounding it, and its minute but practically irresistible movements do not affect its stability, in which, by reason of its immense weight, the effect of passing trains is likewise a negligible factor. The name of General Raymond worthily stands at the head of the list of engineers, on the great memorial tablet at the portal of the Pennsylvania Railroad Station in Seventh Avenue, New York City.

These labors were performed under difficulties which might well have discouraged a less intrepid spirit. General Raymond had lost

almost entirely, some thirty years before, possibly as the result of the hardships of his explorations in Alaska, the use of one eye; and, about 1900, the formation of a cataract in the remaining eye threatened him with entire blindness. Under this increasing disability (which finally reached such a point that he had to be personally led to and through the tunnels, and could examine drawings only with the aid of a strong glass, magnifying one spot at a time), he continued the active and efficient discharge of his duty as head of the directing Board of Engineers until, in 1910, its work was practically done, and his office in New York was closed. Even after that, in his seaside cottage near Atlantic Highlands, N. J., he dictated, at the request of the Pennsylvania Railroad Company, and under great difficulties of ever-growing blindness, a report of the work of his Board, which has been published by the Company and constitutes a memorable contribution to the literature of that branch of engineering.

Meanwhile, many successful operations (each partly, but not decisively, successful) had been performed on his eye, in which the obscuring film repeatedly gathered. In October, 1912, after months of total blindness, he went to Washington once more for another such operation, with more than usual expectation of permanent relief. The surgeons at Washington, however, discovered another and previously unsuspected trouble—an internal malignant tumor which forbade the expected operation and was itself beyond cure. There was nothing left on earth to him but hopeless darkness and cruel pain, which he bore with characteristic fortitude and patience until his death on May 3d, 1913. Conscious and serene to almost the last moment, he crowned with a heroic death a long, active, useful, and distinguished life, graduating at the end, as he had graduated at the Military Academy nearly half a century before, "at the head of his class." The Alumni of West Point may well be proud of his stainless and illustrious record.

Charles Walker Raymond was elected a Member of the American Society of Civil Engineers on June 1st, 1892.

BAIRD SNYDER, Jr., M. Am. Soc. C. E.*

DIED JULY 9TH, 1913.

Baird Snyder, Jr., was born at Pottsville, Pa., on November 21st, 1868. He was the third son of Baird and Edith Morris Snyder. The mother, now deceased, was a great-granddaughter of Robert Morris, so distinguished in the days of the American Revolution and a signer of the Declaration of Independence. His father, Baird Snyder, was the son of George Washington Snyder, one of the early pioneers of

*Memoir prepared by Mr. Frank A. Hill, Gen. Mgr., Maderia, Hill & Co., Inc., Pottsville, Pa.

the Anthracite Region, and the three generations have been prominent in the development and progress of mining in the district.

On his graduation from the Pottsville High School in 1885, Mr. Snyder entered the service of the Pennsylvania Railroad Company as Clerk in the office of the Chief Engineer.

In 1888 he joined the Engineer Corps of the Philadelphia and Reading Coal and Iron Company, and began the engineering career of which his friends and associates are so justly proud and which broadened in experience and grew in accomplishment until his valued life was suddenly closed.

In 1893, the late Joseph S. Harris, while President of the Lehigh Coal and Navigation Company, recognizing Mr. Snyder's ability, appointed him Assistant Superintendent of the mining operations of that Company at Lansford, Pa. Three years later he was made General Manager of the Company, and remained in this position until January, 1912.

The property which came under his management is one of the oldest and best in the Anthracite Region. The various adverse conditions which arise in the experience of the anthracite mine manager were all present during Mr. Snyder's administration at Lansford. The many difficulties of mining, pumping, ventilation, labor, fighting fire under ground, and countless others, were met with patience and self-reliance and were successfully overcome. Among Mr. Snyder's most marked qualifications for his position was his ability to handle men. No man had more loyal subordinates to aid him, and no men ever had a mort intrepid leader.

After nineteen years of active and aggressive work at Lansford, he resigned from his position with substantial expressions of confidence and regard from his Board of Directors and with the heartfelt regret of his subordinates.

On leaving the Lehigh Coal and Navigation Company, Mr. Snyder, as President and General Manager, organized the Locust Mountain Coal Company, for the purpose of operating an undeveloped area of coal land owned by the Stephen Girard Estate. He went into this enterprise with his characteristic fearlessness and vigor. He had completed his financial and working plans, and was in the midst of their early development with every confidence of success, when death came.

He had met and conquered the many engineering difficulties that had come to him in his old work, and was looking forward with pleasure to solving the new and less complicated questions which would meet him in opening an undeveloped territory of his own selection, but on July 9th, 1913, from injuries received in an automobile accident on the preceding day at Wapwallopen, Pa., this strong and virile life, filled with the promise of many more successful years, was cut off

"in the twinkling of an eye", leaving to his family and his many friends and professional associates only the memory of an active and well spent life, full of achievement; the record of an honest man, a good citizen, an able engineer—a record creditable to the membership of any engineering society.

Mr. Snyder was a man of splendid physique, of fine mentality, a reader, and a student. Positive, even to brusqueness, without fear, and a natural leader, he was also a devoted son, a loving husband, and a kind father. He is survived by his widow, Jennie Craig Romig Snyder, and two sons, Baird Snyder, 3d, and Robert Morris Snyder.

Mr. Snyder was elected a Member of the American Society of Civil Engineers on March 2d, 1904.

JAMES DYNAN NEWTON, Assoc. Am. Soc. C. E.*

DIED AUGUST 8TH, 1912.

James Dynan Newton was born at Oswego, N. Y., on April 17th, 1871. He entered Holy Cross College, Worcester, Mass., in 1887, and was graduated in 1891 with the degree of Bachelor of Arts. He taught sciences in the Roman Catholic High School, Philadelphia, for one year and then entered Sibley College, Cornell University, and received the degree of Mechanical Engineer in 1895. In the same year he was granted a Master's Degree from Holy Cross College.

After his graduation from Cornell he served as Special Apprentice in the New York Central Railroad shops at Oswego for five months. In December, 1895, after passing the examinations, he was appointed Cadet Engineer in the United States Revenue Cutter Service, and in 1896 he was promoted to Third Lieutenant of Engineers. He served in this capacity till 1902, when he was retired for disability incurred in the line of duty. He was in the Revenue Cutter Service during the Spanish-American War, and for a part of this time was Acting Chief Engineer of the U. S. S. *Hamilton*.

He was sent to the Marine Hospital at Fort Stanton, N. Mex., for treatment, and after leaving there was employed on mining development in New Mexico, practiced engineering independently, and, in 1904, was employed as Chainman and Rodman on the Atchison, Topeka, and Santa Fé Railroad. In August, 1905, he accepted the position of Assistant Professor of Civil Engineering in the University of Kansas.

The writer was in charge of the Department of Mechanics during the time of Professor Newton's service in the University of Kansas,

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*Memoir prepared by Herbert A. Rice, Assoc. M. Am. Soc. C. E.

and, as a part of his teaching was in this Department, it gave good opportunity for observing his work.

In the Engineering Profession, no doubt, one of the greatest virtues is that of hard, conscientious labor, and this Professor Newton possessed in a marked degree. Entering a new field, he at once saw the necessity of careful preparation. He not only prepared himself for the work in hand, but spent many hours in the study of related subjects. His broad college training and his practical experience enabled him to present his subjects in an interesting and practical manner, and the writer always found his students enthusiastic in his praise.

At the end of his fourth year at the University of Kansas, he was elected Dean of the School of Engineering of Loyola University, Chicago, Ill. Here, his great capacity for labor stood him in hand. The Engineering School which had not been established, had to be organized, courses laid out, catalogue prepared, equipment purchased for a new building then in process of construction, and instructors engaged. This entire work fell to Professor Newton. Success attended his efforts, and, at the time of his death, the school was thoroughly organized and had enrolled many students in the new Department.

Professor Newton was an enthusiastic worker in the Roman Catholic Church, being one of the leading soloists in the church choir while in Lawrence.

He was married in 1906 to Miss Minnie Medaris, of Kansas City, who, with one child, survives him. They reside at Lawrence, Kans.

Professor Newton was elected an Associate of the American Society of Civil Engineers on September 5th, 1911.

ORLOFF LAKE, Jun. Am. Soc. C. E.*

DIED OCTOBER 21ST, 1913.

Orloff Lake, the eldest son of Duff G. and Mary Ida (Woods) Lake, was born at Baltimore, Md., on April 19th, 1883.

He was graduated from Tulane University with the degree of Bachelor of Engineering in 1905, and immediately thereafter entered the service of the Philadelphia and Reading Railway as Rodman, which position he held until April, 1906, when he resigned to accept the position of Transitman with the New York Central and Hudson River Railroad. He was soon promoted to Assistant Engineer, and had charge of office and field work on construction at Brewster; he also made surveys for

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

the relocation of the New Jersey Shore Line from Weehawken to Shadyside.

In January, 1908, Mr. Lake left the New York Central and joined the engineering forces of the writer as Assistant Engineer on general work in and near New Orleans. He remained in this employ until the summer of 1909 when he temporarily withdrew from the engineering field and transferred his activities for two years to the manufacturing business conducted by his father and uncle.

His love for his profession, however, led him to accept employment, in June, 1911, with the Barrett Manufacturing Company, and he was especially engaged in the Tarvia Department to co-operate with county and other engineers in the South who were devoting their time and talents to highway construction. Mr. Lake's education and training especially qualified him for this work, in which he continued with marked success until his sudden and untimely death.

Mr. Lake was a young man of exceptional attainments and brilliant promise. He was a tireless student and a most industrious worker, whose physical strength was without doubt often overtaxed by the tasks which he set for himself to perform. His exceeding modesty, amounting almost to diffidence, no doubt operated in retarding his professional progress during his brief life. His high ideals, sterling good qualities, and quiet forcefulness, however, were such that, as acquaintance with him ripened, confidence in and affection for him grew strong. In his death, the Profession and the Society have lost a most promising member; and in the zone of his acquaintance there is a void which may not easily be filled.

Mr. Lake was elected a Junior of the American Society of Civil Engineers on December 1st, 1908.

HENRY A. RICHMOND, F. Am. Soc. C. E.*

DIED MAY 10TH, 1913.

Henry A. Richmond was born in Syracuse, N. Y., on August 3d, 1840. He was the second son of Dean Richmond who, at the time of his death in 1866, was President of the New York Central Railroad Company.

Mr. Richmond was engaged in the grain business, as a commission merchant, at Buffalo, N. Y., and was well known among shipping circles on the Great Lakes. Later, he was connected with a lithographing business.

*Memoir prepared by the Secretary, from information supplied by C. M. Morse, M. Am. Soc. C. E.

After his retirement from active business, he became prominent in public affairs in Buffalo, as an "old-line Democrat". He was President of the Buffalo School Association, and was appointed by the late Grover Cleveland, then Governor of New York, as a Member of the first Civil Service Commission of New York State.

Mr. Richmond never married. He was a "wide traveler and a devotee of art and literature", and was liked and respected by all who knew him. He died at Los Angeles, Cal., on May 10th, 1913, and was buried in the Richmond Mausoleum at Batavia, N. Y.

Mr. Richmond was elected a Fellow of the American Society of Civil Engineers on July 7th, 1870.

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PAPERS AND DISCUSSIONS

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AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

By John C. Koch, Assoc. M. Am. Soc. C. E.

To be Presented March 4th, 1914.

Introduction.

The writer has been investigating the subject of sand-clay road construction for the past 2 years, for the purpose of establishing a sound basis for the selection of suitable materials for use in this cheap type of road. The materials are so cheap and distributed so abundantly that the writer is convinced that, if the Engineering Profession realized more fully the many advantages of such roads, there would be a much wider adoption of this type of construction.

The purpose of this paper is to set forth the results of the writer's study of these materials, both in the field and in the laboratory. This work was done under the auspices of the University of Georgia, a Good Roads Department having been established in the fall of 1911 for the purpose of carrying out a policy of University Extension work by offering free engineering assistance to the road officials of the State on highway and bridge work. The writer makes grateful acknowledgment of the facilities placed at his disposal by Chancellor

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

D. C. Barrow, of the University of Georgia, and C. M. Strahan, M. Am. Soc. C. E., for many helpful suggestions.

The growing demand for improved country roads at a low enough first cost to permit the construction of a relatively large mileage in counties of average financial resources has led, in many scattered sections of the United States, to an extensive adoption of the sand-clay type of construction.

Sand and clay are the most widely distributed materials in Nature, and, in suitable mixtures, yield a road-surfacing material which can be secured at low cost, because so readily accessible, and when properly built of intelligently selected material, it will render excellent service. Briefly stated, the advantages of sand-clay mixtures for road surfacing are:

(1) Low first cost,
(2) Low maintenance,
(3) No expensive machinery for construction or repairs,
(4) No skilled labor required.

Mixtures of sand and clay in proper proportions for road surfacing often occur in Nature, though good results can be secured by making artificial mixtures. The name "topsoil" has been widely used in Georgia and other parts of the South to indicate a natural mixture of sand and clay found on the surface of the ground over wide areas in the Appalachian region. Such natural mixtures, doubtless, are to be found in many other parts of the United States.

The principal advantages in using natural mixtures of the proper proportions, as compared with artificial ones are:

(1) The expense of mixing is eliminated,
(2) The natural mixtures become compacted in much less time,
(3) Repairs are more easily made, and new materials unite more quickly to the old.

The firmness of the wet sand road and the smoothness of the dry clay road are combined in the sand-clay type, and it remains of almost uniform smoothness throughout the year. It has been known for years in many localities that there were stretches of earth road that always kept hard and smooth, shed the water, and scarcely ever needed any repairs. Fig. 2 illustrates a section of natural sand-clay road near Center, Ga., which has been in use for 60 years. In such places

the surface soil has been found to contain the proper proportions of sand and clay, which, remaining undisturbed, has become compacted into a hard, water-proof surface.

There are many miles of sand-clay roads in Georgia which have cost less than $500 per mile for the surfacing in place. They carry heavy country traffic, and the repairs, during a period of 5 years, have not averaged $5 per mile per year. Considering the fact that the first cost of construction of such roads is less than the annual interest and maintenance charges on almost every other type which has been offered as an improvement on the earth road, it seems inevitable that wherever sand-clay materials are available and of suitable quality, it is the best type to use for the average country road.

Objects of the Investigation.

The writer's objects were to determine:

(1) The proper proportions of sand and clay in sand-clay mixtures to yield the best results, namely, durability in all weathers and absence of mud and dust;

(2) The limits between which the proportions of the two materials could vary without affecting seriously the quality of the resulting road surface;

(3) The effect of varying proportions of different sizes of sand in such mixtures;

(4) An approximate field method for the examination of soils, to determine their suitability for road surfacing.

Basis of the Investigation.

It will be readily seen that, to carry out an investigation along these lines, using artificially proportioned mixtures on short lengths of experimental road, would not only involve large expense, but probably would not be conclusive because of the difficulty of making artificial mixtures according to given proportions and maintaining such proportions. Then again, materials tracked on the experimental sections by the traffic would tend to affect greatly the value of such sections. For these reasons it was determined to base these studies on roads already constructed in various parts of Georgia, and in service from 1 to 5 years. In the course of this work the writer has traveled over 2 500 miles of roads, selecting various sections of sand-clay (including

"topsoil") roads which had been in use for some time and had worn well, and concerning which definite information could be secured as to materials used, methods of construction, and behavior of the road in all seasons of the year. From the best sections selected, samples of the road surfacing in actual service were chopped out with a hatchet for a depth of 4 in. in the wheel-tracks. Usually, a 5 by 5 by 4-in. sample was taken for laboratory study.

Samples were also taken from sections where such roads had not given entire satisfaction, on account of dust and mud. The writer has examined more than 900 samples, representing 48 counties, and comprising every typical sand-clay section of the State.

Properties of the Materials.

Clay.—The properties of a clay depend on the kaolin minerals and the impurities it contains. The kaolin particles may consist of prismatic crystals, thin plates or scales, or both. The effect of the impurities depends on the quantities of each and its fineness. Clays have no characteristic color; the pure are white, but they commonly occur as blue, gray, red, yellow, green, and purple, due to impurities.

All clays are derived from the weathering of feldspar, and the particles vary in size from small pebbles to such a minuteness as to defy measurement by the most refined methods. The finer particles readily go into solution in water, and even the larger ones are easily moved by running water. When removed in this manner and deposited at a distance, the deposits formed are called sedimentary clays. Those formed in place by the weathering of feldspar are called residual clays. In general, the sedimentary are finer grained and more plastic than the residual clays. The writer's experience has been that the latter make a harder and tougher mixture for road work than the sedimentary clays when combined with sand.

Clays vary greatly in density, plasticity, and size of grain. The important characteristics of this material for sand-clay construction are plasticity, shrinkage, and the property of "slaking". The plasticity is indicated by the ease with which a clay when wet to a certain extent can be moulded into various shapes which will be retained after the material has dried. The drying out of clay produces a shrinkage which seems to consolidate the particles into a compact mass. The extent of the shrinkage will depend on the fineness of the clay particles.

The coarser clays and those with much impurity (sand, etc.) shrink but little. The purer clays show a tensile strength of from 50 to 200 lb. per sq. in. when thoroughly dried; the coarser and impure ones have very little tensile strength. The most plastic clays resist water for a long time, but the others often crumble to pieces like quicklime. This rapid breaking down is due to the absorption of water by the pores of the clay, sometimes so quickly as to act with almost explosive suddenness. It is evident that a slaking clay would tend to break down rapidly into mud when exposed to rain and the puddling effect of traffic.

Sand.—Sand consists of particles of various sizes formed by the disintegration of the many kinds of rock in which quartz appears as an important component. As ordinarily found, the individual particles possess great crushing strength, as well as resistance to abrasion. These two qualities are of the greatest importance in the sand-clay combination.

Silt and Organic Matter.—In most of the natural sand-clay mixtures occurring in proper proportions for use directly as a surfacing material, the quantity of silt is usually from 3 to 8 per cent. The quantity of organic matter varies considerably, but, as it is light and easily washed out of the road by rains, its only effect seems to be to hold a certain quantity of moisture beneath the surface. Silt, if not in too large a quantity, seems to improve the road. In a few cases the writer has found silt and organic matter together, in quantities as great as 20% of the total sand-clay mixture, and yet the road was good. Probably in such cases a large quantity of water is held by this porous material and this aids in binding the sand particles together. Thus it appears that these materials are not necessarily harmful to the mixture.

Mixtures of Sand and Clay.

Whether natural or artificial mixtures are used, the properties of both materials which are best adapted to resist the weather and the wearing effect of traffic are utilized, and the unfavorable qualities of both are largely eliminated by using the two in proper combination. It has been shown that clay in the pure state shrinks as much as 10% when dried, and on being wet again will expand to an equal extent. By mixing clay with the proper quantity of sand, these contractions and expansions may be eliminated almost entirely. There-

fore, in a sand-clay mixture for road work, the sand, being held together by the clay, which usually more than fills the voids in it, takes the wear of the traffic in dry weather. In wet weather the water quickly drains off this almost impervious mixture, and the sand resists the cutting action of the traffic, although the clay may become puddled to a certain extent.

The writer is aware of the difficulties and intricacies introduced into the subject by the wide variety of clays available and their different characteristics. Nevertheless, he believes that the methods given in this paper, which have thus far proved highly satisfactory in the selection of sand-clay materials in many parts of Georgia, will be of general value wherever such materials are to be found. This basis of study of these materials has been of great service in choosing the best of a number of equally accessible deposits of natural sand-clay mixtures. It may with equal facility be used for determining in just what proportions the different materials should be used in order to get the best mixture possible out of the available local deposits where it is necessary to make artificial mixtures. Most clays contain varying proportions of sand, even when they appear to be almost pure. Clays are often sent in from the southern part of Georgia (where the surface soil is very sandy and it is desirable to lay clay on top of it in order to mix with it and form a sand-clay surface), and on analysis are found to contain as much as 60% of sand.

Laboratory Procedure.

Equipment.—So much that is useless is often purchased for laboratory equipment that the following list is given for the benefit of those who seek definite information on this subject. These have been found to give quite satisfactory service, and need not cost more than about $50.

One metric balance, 111-gramme capacity, reading to 0.01 gramme.
One metric scale, 2 500-gramme capacity, reading to 5 grammes.
One set of sand sieves, nesting, 8 in. in diameter, Nos. 10, 20, 40, 60, 80, and 100, with dust pan and lid, all brass.
Two steel moulds, brass lined, 1 in. in diameter, 4 in. long, with 5-in. close-fitting plunger.
One wooden mallet.
Six 300-cu. cm. evaporating dishes, porcelain.

Two Bunsen burners.
Several small tripods for burners.
Several iron wire mats, 4 by 4 in., for Bunsen flame.
Thermometer, 0° to 150° cent.
Six, 2-quart, enameled milk pans for baths, etc.

The evaporating dishes should be Royal Berlin porcelain, as this is the only kind that the writer has found satisfactory.

Separation of Materials.—Samples are usually taken in several parts, so as to enable one to determine to what depth the material may be used. In some cases a sample may be in three or more parts, being 4-in. layers taken in succession from the same hole, each being kept separate from the others. After analyzing each part of a sample separately, the composition of the top 4, 8, 12, or 16 in. can be determined easily. This gives a clear idea of the value of the material and of the greatest depth to which it may be excavated.

The essential objects of the laboratory examination are to ascertain:

(1) Relative proportions of sand and clay,
(2) Physical analysis of sand,
(3) Physical properties of the mixture of sand and clay,
(4) Determination of presence of other material which might prove detrimental,
(5) Physical properties of the clay.

To attain these objects, the following steps are necessary, in the order of their importance, and to reduce the laboratory work to a minimum:

(1) Separation of sand and clay,
(2) Mechanical analysis of the sand content,
(3) Slaking test on cylinder of sand-clay,
(4) Examination for mica and feldspar,
(5) Slaking test on clay cylinder.

1.—Separation of Sand and Clay.—The sample to be examined is first dried thoroughly in the air; then it is screened through a No. 10 sand sieve (10 meshes per linear inch), lumps and clods being pulverized with a wooden mallet beforehand. All coarse materials caught on the No. 10 sieve are considered arbitrarily as gravel, and all smaller materials are taken as the sand-clay portion. Pieces of grass, roots, etc., are discarded, of course, before weighing the two parts of the

screened sample. The material held on the No. 10 sieve is regarded as being much the same as the broken stone in a concrete mixture, and the sand-clay as the cement and sand portion constituting the mortar in such a mixture.

The part of the sample passing the No. 10 sieve is then examined as follows: By successive quartering, about 150 grammes are taken and dried in the air bath at 100° cent. to constant weight. This may often be dispensed with, and the sample merely air dried. Then 100 grammes of this material are weighed and placed in a porcelain evaporating dish; water is added and the soil rubbed well until the clay particles are in suspension. The clay in suspension is then carefully poured out and more water is added; the process is repeated until there is faint or no coloration of the water when the residue is stirred up. By washing the materials properly, practically all the organic matter, silt, and clay are removed, and only the sand particles are left, with possibly some mica and feldspar.

2.—Mechanical Analysis of the Sand.—The moisture in the residue in the evaporating dish is then evaporated, leaving the sand dry and ready to be screened. After weighing the sand it is screened through a nest of five sieves having 20, 40, 60, 80, and 100 meshes per linear inch. The weights of sand caught on each of these sieves are determined, and a curve is plotted showing the weights as ordinates and the sieve sizes as abscissas, based on 100% for the total weight of the mixture of sand and clay. Thus the weights in grammes show as percentages of the total. Fig. 1 was prepared in this way, and shows graphically the sand analyses of a number of samples.

3.—Soil Slaking Test.—A test cylinder, 1 in. in diameter and 3 in. long, is made of the material passing the No. 10 sieve by wetting it sufficiently to make a very stiff paste and after working it thoroughly together, placing it in a metal mould, using a tight-fitting plunger to compact the material as much as possible by tamping with a mallet. This cylinder should be dried to constant weight in the air bath at 100° cent. When entirely cooled it should be immersed completely in a glass jar of water at a temperature of 21° cent., and the time noted in which it disintegrates completely. Disintegration is assumed to be complete when the cylinder has broken down until the material is standing approximately at its natural slope of repose.

In sand-clay mixtures which have given satisfactory service, the

time to disintegrate completely may vary from 2 min. to nearly 1 hour; usually, it will be from 5 to 20 min. This test will give a fairly good idea of the resistance of any sand-clay mixture to the action of water, and, for purposes of comparison, is made more easily and

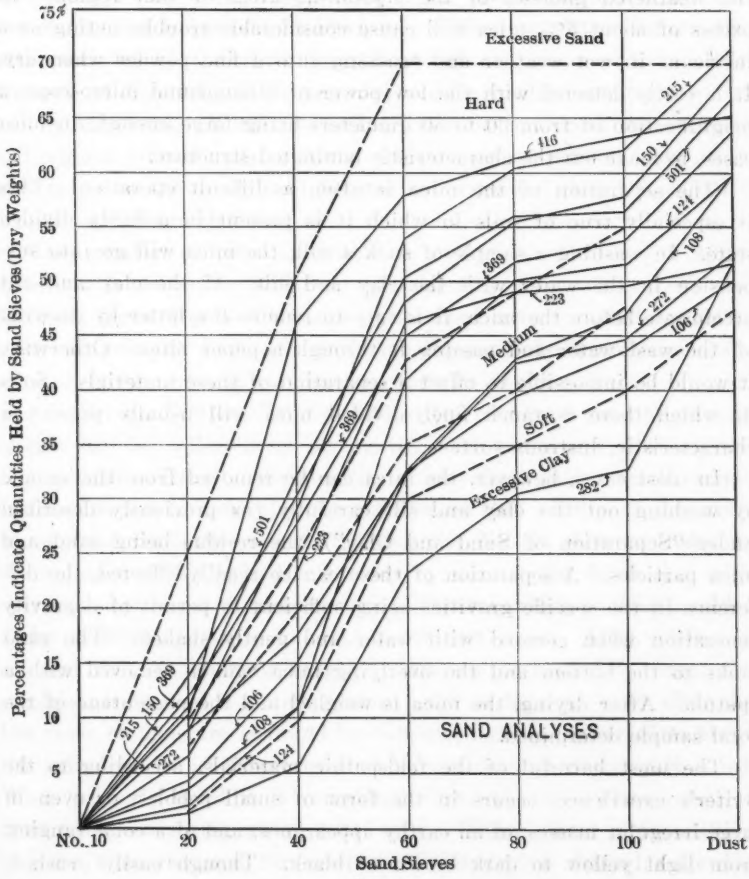


FIG. 1.

quickly than that to be described for the slaking of clay alone. The most durable mixtures, in general, are those which take longest time to disintegrate. Mixtures containing a coarse, flaky clay may hold their shape for several days before breaking down completely, but this is not a frequent occurrence.

4.—Tests for Mica and Feldspar.—Mica is easily recognized, and its presence need occasion no alarm unless it exceeds 5% of the total sand-clay sample. This is especially true of the Southern Appalachian region, where mica is often associated with the coarser sands, as in the weathered gneisses of the crystalline areas of that region. In excess of about 5%, mica will cause considerable trouble, acting as a lubricant in wet weather and crushing into a fine powder when dry. It is easily detected with the low power of a compound microscope, a magnification of from 30 to 50 diameters being large enough, in most cases, to make out the characteristic laminated structure.

The separation of the mica is often a difficult operation. This is especially true of soils in which it is present in a finely divided state. In washing a sample of such a soil, the mica will go into suspension in the water with the clay and silt. If the clay and silt precipitate before the mica, it is easy to remove the latter by drawing off the wash-water and passing it through a paper filter. Otherwise, it would be impossible to effect a separation of these materials. Soils in which there is much finely-divided mica will usually present a characteristic, lustrous surface.

In most cases, however, the mica can be removed from the sample by washing out the clay and silt carefully (as previously described under "Separation of Sand and Clay"), the residue being sand and mica particles. A separation of these can be readily effected, the difference in the specific gravities being sufficient to permit of a gravity separation when covered with water and gently shaken. The sand sinks to the bottom and the overlying mica can be removed with a spatula. After drying, the mica is weighed and the percentage of the total sample determined.

The most harmful of the feldspathic materials, according to the writer's experience, occurs in the form of small pebbles, or even in large irregular masses, of an earthy appearance, and of a color ranging from light yellow to dark brown or black. Though easily crushed, it will generally have more coherence than the clods of earth or clay in the sample. Therefore, the larger pebbles can be separated by screening the dried sample through the No. 10 sieve. If pebbles of other material are also found, the difference in color will permit of a separation. The feldspathic materials passing the No. 10 sieve, together with the sand, clay and silt, will be left with the sand, as a

residue after washing out the other materials. Separation of the feldspar can then be made in the manner described for mica. If this material occurs in greater proportions than about 8% of the total sample, by dry weight, the road surface will cut and wash easily. This is a figure based on actual experience with a few roads, and is probably subject to modification, depending on the characteristics of the other components of any given sample. The presence of fine feldspathic material will be readily apparent in the soil cylinder test. When the soil cylinder is immersed the feldspar will almost immediately go into solution or suspension, and in a minute or so the water will become so discolored that the cylinder cannot be distinguished.

The number of cases in which a separation of either of these materials is advisable will vary considerably. In the writer's experience, about 6% of all samples examined were analyzed quantitatively for mica or feldspar. Although the methods described are approximate, they have given very satisfactory results.

5.—Slaking Test on Clay Cylinder.—The clay from the 100-gramme sample can be collected by saving the wash-water and allowing the clay to settle. Usually, the quantity of clay from such a small sample is not sufficient to make the test cylinder, so that additional clay must be removed from the material passing the No. 10 sieve to make up the cylinder. This cylinder is made by mixing the dried clay with only sufficient water to make a very stiff paste, which is then moulded under sufficient pressure to make a compact mass. After drying in air and then to constant weight in the air bath at 100° cent., the specimen is kept at that temperature until at least an hour after the last trace of moisture is found by condensation on a glass plate above the cylinder. It is important that the cylinder be thoroughly dried. When properly dried it is completely immersed in a glass vessel of proper size containing water at 21° cent. The character of the slaking and the time required for complete disintegration depend somewhat on the temperature of the water, and the freedom from moisture of the dried cylinder, so that, for a comparison of results, all specimens should be thoroughly dried and the water for the test brought to the same temperature in each case.

The slaking of clay is similar to that of quicklime, except that the action is entirely physical and not chemical. The time and character

of the slaking give an idea of the value of the clay as a binder in the sand-clay mixture. In clays which have been separated from samples taken from road surfaces which have proved satisfactory, the time for the clay cylinder to disintegrate completely varies from 2 to 20 min., the average being about 4 min. The degree of disintegration of the clay may vary from finely divided particles to flakes almost $\frac{1}{8}$ in. long. In general, the coarser the particles the better the indication for suitability for road work. Clays which disintegrate completely in less than 2 min., may be regarded with some suspicion, but need not necessarily be rejected entirely, unless the sand analysis and the soil cylinder also give poor tests.

Typical Examples, with Analyses and Road Histories.—It is thought that a few characteristic cases from actual practice may be of interest. In Table 1 is given a list of the samples, with the locality from which each was taken. Table 2 shows the sand and clay analysis, based on the dry weight of the sample, the total weight of each sample being taken as 100%; all sand percentages are based on this.

TABLE 1.—TYPICAL SAND-CLAY ANALYSES.

Percentages of Total Weight of Sand and Clay.

Sample No.	106	108	124	150	215	223	272	369	416	501	282
Sand No. 10	(12.5)	....	....	(6.3)	....	(34.0)	....	(7.2)	(1.5)	(10.0)	....
" No. 20	4.5	3.6	2.0	13.7	9.2	6.8	9.2	12.0	10.6	11.0	1.4
" No. 40	10.8	7.6	9.0	16.6	19.5	15.9	8.6	15.7	35.4	21.4	4.3
" No. 60	17.5	21.5	30.0	17.7	25.2	23.5	14.5	17.3	11.9	17.8	17.5
" No. 80	6.7	10.2	9.5	7.5	6.5	2.8	10.3	6.8	3.3	5.8	7.3
" No. 100	4.0	4.0	1.0	2.3	1.6	0.4	2.0	2.0	2.1	0.4	2.1
" Dust	6.5	13.0	8.5	11.5	9.5	2.1	8.0	10.0	2.0	10.4	19.1
Total Sand	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Clay	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3
Sand 20-60	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2

NOTE.—Sand No. 10 is based on gross sample = 100%, and includes all gravel. Remaining portion of table is an analysis of material smaller than No. 10.

Fig. 1 shows a graphic analysis of these samples, with the sand analysis in detail. This diagram also gives, graphically, the limits between which the writer classifies his samples, basing such classification on the sand analysis as well as the clay content, both of which are shown.

The limiting curves separating the analyses into four groups are based on the writer's experience, and are given with full confidence in their utility. They are of sufficient accuracy to be of considerable

service in classifying sand-clay mixtures properly and quickly, and arriving at a reasonable approximation of their value as road material. Quite often it will be found that the sand analysis alone may show that the material is unsuitable and much time may be saved that might otherwise have been lost in continuing the examination and applying other tests.

TABLE 2.—ROAD HISTORIES.

Sample Numbers Refer to Corresponding Numbers in Table 1.

Sample No.	County.	Road.	Sample taken.
106	Bulloch	Statesboro-Savannah	6.7 miles south of Statesboro.
108	Dougherty	Albany-Thomasville	4.3 miles south of Albany.
124	Sumter	Americus-Albany	6.0 miles south of Americus.
150	Clarke	Athens-Danielsville	3.2 miles northeast of Athens.
215	Dougherty	Albany-Thomasville	2.1 miles south of Albany.
223	Richmond	"Augusta gravel" from pit.	
272	Habersham	Clarkesville-Cornelia	1.2 miles from Clarkesville.
369	Clarke	Athens-Barnet Shoals	4.3 miles from Athens.
416	Hall	Thompson's Bridge	2.0 miles north of Gainesville.
501	Clarke	Athens-Whitehall	2.0 miles from Athens.
282	Decatur	Bainbridge-Jacksonville	7.0 miles south of Bainbridge.

The analyses falling in the group marked "Hard" will usually give a durable, hard road surface which wears exceedingly well, and, after consolidation, can be cut only with great difficulty by a road machine, if at all. Analyses in the "Medium" group will give an excellent, smooth, hard, surface, but one which may cut a little in protracted wet weather. Such a surface is too hard to attempt to shape up with a road machine, except after very long wet periods. This may be taken as a sort of "average" quality of sand-clay.

The analyses grouped in the "Soft" class give a surface which is much superior to the ordinary earth road, but which will tend to be somewhat dusty in dry weather and will cut easily and tend to wash in wet weather. Material of this type requires re-shaping, after every heavy rain, with a road machine or drag. The class "Very soft" comprises all materials which have too large a clay content to give a satisfactory road surface; they should not be used at all, as the expense is scarcely warranted for the kind of road surface that can be obtained with it.

The writer's method is to make the separation of sand and clay and then the mechanical analysis of the sand. If the material shows up poorly in the sand analysis, the further tests are not made, unless,

after making a similar examination of all other available materials, it appears to be the best available. Each case is a special problem in economy, and a study of each will indicate the proper method of using local materials if they can be used at all.

Road Histories.

The following is a brief history of the various roads, samples of which were analyzed and the results presented in Table 1, and graphically in Fig. 1:

106.—Bulloch County.—Had been in service 3 years at time sample was taken. Excellent, hard, smooth surface, but occasionally dusty in prolonged dry seasons. Softens in wet weather and cuts badly then. Re-shaped after heavy rains with light drags. Artificial mixture.

108.—Dougherty County.—Good smooth surface in dry weather. Road in service about 3 years at time sample was taken. Softens and washes in heavy rains. This material, found about 2 ft. below natural surface, is used without addition of other material.

124.—Sumter County.—Very good, smooth surface, but softens and washes badly in wet weather. Re-shaped with light drags after even moderate rains. This is generally true of all the roads in this county, the materials available being very similar in mechanical analyses. Natural mixture occurring 2 ft. below surface.

150.—Clarke County.—Road in use 4 years, with practically none but the very lightest repairs, when sample was taken. Cost of maintenance, as given by superintendent, averaged less than $5 per mile per year. Very little dust, firm, hard road surface throughout the year, though freezing and thawing might at times soften the crust for a depth of an inch or so. No re-shaping with road machine possible, on account of the hardness of the compacted material. This seems to represent the best and most durable type of sand-clay combination yet found in Georgia. Natural mixture found as topsoil of cultivated fields.

215.—Dougherty County.—Road in use about 3 years at time sample was taken. Good hard surface, softens but little in wet weather, and wears well, keeping good shape. Too hard to re-shape by dragging. Few repairs. Natural topsoil.

223.—Richmond County.—Reports of roads on which this material had been used indicate that it produces very satisfactory results. This

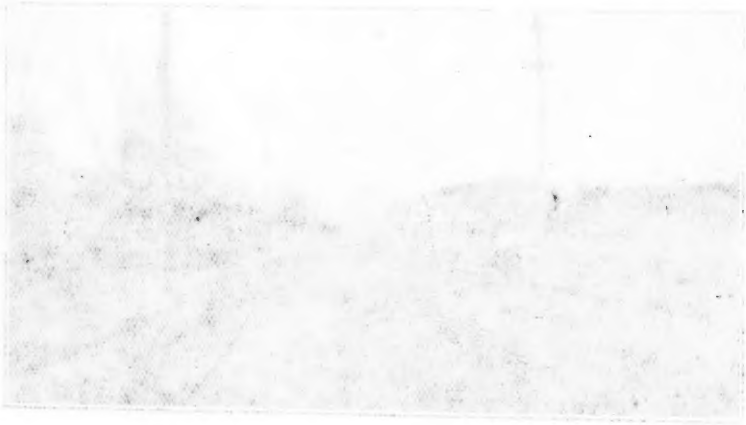
FIG. 2.—NATURAL SAND-CLAY ("TOPSOIL") ROAD, NEAR CENTER, GA. IN USE ABOUT 60 YEARS.

FIG. 3.—NEAR THE SAME PLACE AS SHOWN BY FIG. 2. NEW ROAD ON THE RIGHT.

FIG. 4.—TYPICAL SAND-CLAY ROAD IN THOMAS COUNTY, SOUTHERN GEORGIA.

FIG. 5.—TALLASSEE ROAD, CLARKE COUNTY, GEORGIA. TOPSOIL IN PLACE SIX DAYS, WITH TWO DAYS OF HEAVY RAIN.

FIG. 6.—CENTER ROAD, CLARKE COUNTY, GEORGIA. CLAY ROAD AT LEFT. NATURAL TOPSOIL ROAD AT RIGHT.



material is shipped by rail for road construction for distances as great as 100 miles. The analysis in Table 1 indicates that this is really a low-grade gravel rather than a sand-clay mixture. Yet it is interesting to note that the mechanical analysis of the portion of this material smaller than No. 10 mesh agrees very well with the mechanical analysis of the better classes of sand-clay mixtures which, with very small gravel contents, have given results almost as good. The mechanical analysis is shown on Fig. 1, considering only the portion smaller than No. 10 mesh. This material is dug from a large pit near Augusta, and has been in use for many years.

272.—Habersham County.—The road from which the sample was taken had been in use for about 8 months. The surface is hard and smooth, and softens but little in wet weather. This material was selected by the writer after sampling a large number of local deposits. Haul, about 2 000 ft. Cost in place, for 16-ft. width of surfaced roadway, about $500 per mile. Local officials, after using macadam at about $6 000 per mile, pronounce this a better road than macadam, even if initial cost and maintenance were equal. Local natural mixture of sand-clay from topsoil of cultivated field.

369.—Clarke County.—Built about 3 years at time sample was taken. Hard, durable surface. Few repairs. Too hard to drag. Experience quite similar to that of No. 150. Natural topsoil sand-clay mixture, obtained from nearby cultivated fields.

416.—Hall County.—In use less than a year. Replaced macadam road that cost $6 000 per mile, and gives better satisfaction than the macadam. Cost $350 per mile for 16-ft. width of roadway. Material natural topsoil from nearby cultivated field.

501.—Clarke County.—Road from which sample was taken had been in use about 3 years. This road was surfaced with a very thin layer of natural topsoil from adjoining fields, so that, after being compacted by traffic, its thickness was about 3 in. In seasons of prolonged wet weather traffic occasionally cuts through and mud-holes are formed. The material is excellent, but has not been used in sufficient quantity to give the best results. A comparison of its mechanical analysis with that of other samples indicates its high quality.

282.—Decatur County.—The road from which this sample was taken had been in use about one year. This is practically no improvement over the ordinary earth road, as the sand content is so low as

to be of no value in improving the quality of the road surface. It is very dusty in dry and very muddy in wet weather, after the traffic has been on it a few days. As difficult to maintain as an ordinary earth or clay road. Material used was taken from adjoining fields at a depth of 2 ft.

Table 3 is a summary of the road histories and the corresponding analyses, for comparison. For simplicity, the portion of the sample larger than No. 10 will be considered apart from that smaller than No. 10.

Effect of No. 10 Sand.—Comparing Sample No. 106 with the others, it is found to contain a larger quantity of No. 10 sand than any of them, with the exception of No. 223. No. 223 is really a low-grade gravel, and is discussed in a succeeding paragraph. The history of No. 106 is not nearly so satisfactory as that of the remaining samples, except Nos. 108 and 282. Sample No. 108 is slightly more satisfactory than No. 106, and No. 282 is practically an earth road. Furthermore, Samples Nos. 124, 215, and 272, which contain no No. 10 sand at all, have given much more satisfactory service than No. 106.

It has been found by the writer, in samples of gravel and chert roads (Augusta gravel being a typical material), that the part larger than No. 10 varied considerably in the few samples examined, although the composition of the part smaller than No. 10 was fairly constant in the proportion of sand to clay. The gravel part of any sample may reasonably be taken as an approximate measure of the durability of a road built with such mixtures. In addition, if a sample has a considerable proportion of gravel and is deficient in sand, the presence of the gravel will largely offset such defect.

From the foregoing it is clear that the effect of No. 10 sand on a mixture may be, either that of making up a deficiency of sand in the portion of the sample smaller than No. 10, or to increase durability when there is no deficiency of sand. Hence, its effect depends largely on the analysis of the portion of the sample smaller than No. 10.

Material Smaller than No. 10.—Based on their histories, the samples of Table 3 may be divided into four groups:

1. Ordinary earth, No. 282;
2. Inferior sand-clay, Nos. 106, 108, 124, and 272;
3. Superior sand-clay, Nos. 150, 215, 369, 416, and 501;
4. Low-grade gravel, No. 223.

TABLE 3.—Summary of Road Histories and Analyses.

Sample No.	106	108	124	150	215	223*	272*	369	416*	501	282*
Character of Road Service:											
In dry weather	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Medium
In wet "	Soft	Soft	Soft	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Soft
In prolonged weather	Cuts 2"-6"	Cuts 2"-4"	Cuts 1"-4"	Cuts ⅛"-1"	Hard Cuts ⅛"-1"	Hard Cuts ⅛"-1"	Cuts 1"-2"	Cuts ½"	Cuts ½"	Cuts ½"	Cuts 6"-10"
In freezing weather	Cuts 2"-6"	Cuts 2"-4"	Cuts 1"-4"	Cuts ⅛"-1"	Cuts ⅛"-1"	Cuts ⅛"-1"	....	Cuts ½"	Cuts ½"	Cuts ½"	Cuts 6"-10"
Frequency of re-shaping: Times per year	10-20	10-20	15-25	None in 4 yrs.	None in 3 yrs.	None in 4 yrs.	None in 8 mos.	None in 3 yrs.	None in 7 mos.	None in 3 yrs.	2-4
Sand No. 10, percentage	12.5	.....	.....	6.3	.....	34.0	.....	7.2	1.5	10.0	.....
Sands Nos. 20-60, percentage	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2
Total sand, percentage	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Total clay, percentage	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3

Notes:—*Data for No. 223, based on roads built near Augusta, Ga.
No. 272, Road in use 8 months at time of report.
No. 416, Road in use 7 months at time of report.
No. 282, practically an earth road.

Considering only the second and third groups, the sand-clay analyses may be condensed into Table 4.

TABLE 4.—COMPARATIVE ANALYSES OF SAND-CLAY GROUPS.

	Inferior Sand-Clays.		Superior Sand-Clays.	
	Limits.	Average.	Limits.	Average.
Sand total	52.6-60.0%	56.3%	63.8-71.5%	67.6%
Sand Nos. 20-60	32.3-41.0%	36.6%	45.0-57.9%	51.4%
Sand No. 100 and dust	10.0-17.0%	13.5%	4.1-13.8%	8.9%
Clay total	40.0-47.4%	43.7%	28.5-37.0%	32.7%

The principal differences between the two groups are:

1. The total sand content in inferior mixtures averages about 11% less than in superior mixtures;
2. The total quantity of sand from Nos. 20 to 60, in the former averages 15% less than in the latter.

Effect of Size of Sand.—Although the quantity of clay is nearly the same in Nos. 106 and 282, the former is a much more durable material. Roads built of the former withstand severe rains far better than those built of the latter material. To account for this, it must be remembered that the analysis of the whole sample of No. 106 is: Sand No. 10, 6.5%; sand No. 20 to dust, 43.7%; clay 43.8 per cent. The difference in the clay content is only about 4% in the two samples, which seems too little to account for the great difference in their conduct. By reference to the sand analyses of the two samples, it is seen that No. 106 contains nearly three times as much of the No. 20 and No. 40 sands, nearly the same quantities of Nos. 60, 80, and 100 sand and only one-third as much dust, as No. 282. This considerable difference in the ratio of coarse to fine sands explains, in large measure, the difference in the durabilities of the two materials.

CONCLUSIONS.

A study of nearly a thousand analyses, in the laboratory, and a comparison of the results attained with such materials in actual road construction, have led the writer to formulate the following tentative working rules for examining sand-clay mixtures and as a basis for making up proper combinations for artificial mixtures:

(1) The total relative sand content, disregarding the size of the sand grains, is no criterion of the value of the material.

(2) The sand smaller than No. 60 is of little value in the mixture, that smaller than No. 100, except in very small quantities, is detrimental.

(3) The greater the proportion of coarse to fine sand the harder and more durable will the road surface be.

(4) For the best possible results with sand-clay mixtures, the sand smaller than No. 10 and larger than No. 60 should not be less than 45% nor more than 60%, by dry weight, of the entire sample. In addition, the sand smaller than No. 10 and larger than No. 60 should be composed of about equal parts of Nos. 20, 40, and 60. The total sand content should in no case exceed 70% by weight, of the total sample.

(5) Test cylinders of the sand-clay mixture, 1 in. in diameter and 3 in. long, should, when thoroughly dried in air bath at 100° cent., take at least 2 min., when immersed in water at 21° cent., to crumble down to the natural slope of the material, and preferably should take 6 min. If the cylinder fails in this test, it should be regarded with suspicion. If the sand analysis is poor and the cylinder test is also poor, the material is not worth using.

(6) Test cylinders, made from the clay removed from the sample, 1 in. in diameter and 3 in. long, should take at least 2 min. to crumble down to the natural slope of the material, when immersed in water at 21° cent. If it fails in this test, but passes the test of the preceding paragraph, it may be used, but it indicates a poor quality of binder.

Approximate Field Method for Examining Sand-Clay Mixtures.

As a rapid aid in forming a judgment in the field as to the value of any mixture, the writer offers the following: A buggy inspection is made of the natural soil on both sides of the road to be improved. Samples are taken wherever the surface appearance seems to indicate favorable material. These samples are placed in paper bags, with notes of the location, etc. The samples are usually taken for a depth of 4 in. Each sample can be dried sufficiently by spreading out in the sun in a thin layer. When dried, the sample may be placed in

a No. 10 sieve and screened, and note made of the relative weights of the residue on the screen and the finer material. Then 100 grammes of the part passing this screen are weighed and placed in an evaporating dish. By careful washing, in a few minutes practically all the clay may be washed out, and is rejected. The sand residue is then dried with an alcohol flame, and weighed. It is then placed in a No. 20 sieve nesting over a No. 60 sieve. By screening the sand thus it can be quickly separated into three sizes: No. 20, between Nos. 20 and 60, and smaller than No. 60. These three portions can be weighed, and from this information a very good idea may be obtained of the comparative value of the material, without additional tests.

The proportion of the material coarser than No. 10 indicates the proportion of gravel present. The separation into No. 20 size and between Nos. 20 and 60 shows practically into what class the mixture would fall, and the portion smaller than No. 60 would indicate the quantity of almost worthless material in the mixture. By using Fig. 1 a very good idea of the quality of the sample would be gained.

Methods of Sand-Clay Construction.

There are two general methods of construction in use, one in which the natural mixture of sand and clay is used, the other in which an artificial mixture is made by using two or more materials which are mixed by plowing together, puddling, etc.

Construction with Natural Mixtures.—The method which the writer believes is simplest, and produces excellent results, is as follows: Natural mixtures of the proper proportions of sand and clay may be found as the natural topsoil of cultivated farms, or may be found below the surface at various depths. The material comprising the topsoil of cultivated land, when composed of the proper combination of sand and clay, has probably been more thoroughly weathered and therefore is less likely to wash and disintegrate when placed on the road. Cultivation has also produced a more thorough and complete mixture of the two materials.

In the northern half of Georgia the topsoil in large areas, especially on the tops of ridges, for a depth of from 6 to 12 in., is found to be an excellent sand-clay mixture. In the southern half of Georgia, a natural mixture of sand and clay is often found at a depth of from 2 to 5 ft. below the surface. Such material is not usually very well

JEFFERSON ROAD, CLARKE COUNTY, GEORGIA.

FIG. 7.—TOPSOIL PLACED SIX HOURS BEFORE PHOTOGRAPH WAS TAKEN.

FIG. 8.—TOPSOIL AFTER ONE WEEK OF TRAFFIC, WITH THREE HEAVY RAINS.

FIG. 9.—JEFFERSON ROAD AFTER SIX MONTHS OF TRAFFIC, OCTOBER, 1912, TO MARCH, 1913. TWELVE HOURS AFTER HEAVY RAIN.

weathered, so that, after being placed in the road, a certain quantity is found to wash out quite readily with the first few rains.

In either case the method of construction may be the same. The sub-grade of the roadway is brought to a level or slightly convex cross-section. The sand-clay is then placed in a continuous layer, from 10 to 12 in. thick, the material being spread as fast as delivered and not dumped in piles here and there. This layer is spread for a width of 20 ft. for a nominal 30-ft. roadway. After a sufficient quantity has been placed in this manner, an ordinary road machine is drawn along the ditch line, cutting about 4 in. deep at the outside, and the blade is set so as to cast the material from the ditch against the edge of the sand-clay layer. In this way a shoulder is built up against the sand-clay to hold it in place. This also shapes the ditch. After both sides have been thus shaped, the road machine, in successive passages, rounds up the cross-section of the sand-clay so as to give proper crown to the roadway and a smooth line from the crown to the ditches. As soon as the road is shaped, traffic and the construction teams begin to compact it, and it rapidly becomes consolidated without the use of a road roller. As the consolidation progresses, ruts are formed, and they should be filled and a proper cross-section maintained by the occasional use of the road machine for a period of about 2 months. Unless this is done, the road surface will become rutted and rough, and eventually compacted with a concave crown which will prevent proper drainage. After the material has been consolidated into a hard mass, the difficulty of securing a good cross-section is largely increased.

The cross-section which seems to have given the most generally satisfactory results is a parabolic form with a crown of $\frac{1}{2}$ in. per ft., that is, for a roadway surfaced for a width of 20 ft., the crown would be 5 in., and the height of the center of the road above the ditch (for a road having a width of 30 ft. between ditches) would be 7.5 in. With steeper crowns than this it has been found that the surface cuts into a series of parallel ridges running from the wheel tracks to the ditches and making it very disagreeable for travel. If less crown is given, the provision for wear is too small, and the drainage may not prove satisfactory after a comparatively short time.

For several months rains are apt to soften the top crust and cut up the smooth surface, but if patience is exercised and the road

machine is used to maintain the cross-section properly, it will be found that the puddling action of the traffic when the road softens is a great aid to final consolidation.

Construction with Artificial Mixtures.—From analyses made of materials proposed for use on account of accessibility, and from a study of their sand analyses, the proper ratio in which two or more materials should be mixed can be determined so as to secure the best possible results with the available materials. Three cases arise in which artificial mixtures are to be used:

(1) Sand foundation, where clay is to be hauled and proper mixture made by disk plowing and puddling.
(2) Clay foundation, where sand is to be hauled and mixture made as above.
(3) Soil foundation, where both sand and clay are to be hauled and mixture made.

In any of these three cases the proper mixture of the materials and the puddling action of traffic are necessary to secure a good consolidation. It takes considerable labor to secure a satisfactory mixture, but, except for this, there is no essential difference in the fundamental principles applying to construction with either artificial or natural sand-clay mixtures. The use of the road machine to maintain the cross-section and the height of the crown should be the same for each type of construction. For the softer varieties of sand-clay, the split-log and other forms of light drags may be used effectively in maintenance.

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REPORT ON A SERIES OF TESTS ON CONCRETE COLUMNS REINFORCED WITH A SPIRAL OF STEEL.

By C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie,* and C. O. Carey.*

To be Presented March 18th, 1914.

General Statement.

In 1907-08 the writers were designing some structures in which certain columns were required to carry heavy loads and would be subjected to considerable shock. The nature of the structures was such that columns of reinforced concrete offered certain advantages, sufficient in importance to warrant the use of that material, even though low stresses and consequently an excessive quantity of material might be required to insure safety. At about this time a number of failures of reinforced columns in important structures at various places in the United States were reported in the engineering journals, sounding a warning against a too liberal estimate of their strength, and plainly indicating the need of a careful study of such members from the experimental standpoint as a basis for safe design. Some experiments then made on the ultimate strength of columns with longitudinal steel

* Messrs. Brodie and Carey are Instructors in the University of Michigan.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

bars embedded in the concrete indicated that such reinforcement, unless hooped or banded at frequent intervals, could not be relied on to increase the strength of the column, and in some cases might even render its strength less. These experiments, however, were few in number, and the time available did not permit of their being carried out with sufficient care to render the results thoroughly trustworthy; but they served to turn attention to the hooped column, which had been discussed by Considère in his treatise, regarding which experimental information was very meager.

A series of tests on small columns was planned and partly carried out during the first half of 1908, but the departure of Mr. Wrentmore for the Philippines, in July of that year, interrupted the work, and since that time it has been continued by Messrs. Brodie and Carey at irregular intervals, at such times as their University duties would permit.

In all, 115 columns have been made, and usable results have been obtained from 104 of these; the remaining 11 were broken in storage or in handling, were found faulty in construction, or showed such erratic results under tests as to warrant their being thrown out of the list. Those which will be discussed here are Nos. 10 to 22, and 36 to 115, both inclusive, except Nos. 40, 55, 83, 86, 88, 91, 103, 108, 109, and 112, a total of 83 columns. The other 21, making up the 104, include the columns reinforced with longitudinal rods previously referred to, and some special cases which are not germane to the present discussion. The series is still incomplete, some of the sub-groups showing no tests, or only one or two, and others showing wide variations and requiring more tests to determine a dependable average. Moreover, the extensometer tests on a large percentage of the columns were only carried to 700 or 800 lb., and, later, when a comparison of results was made, it was found that it would have been desirable to have carried all tests to at least 1 400 lb. per sq. in. Some columns gave usable results on only one part of the test, as on ultimate strength alone, or extensometer measurement alone; however, in view of the care which has been exercised in carrying out the work, it is believed that the results obtained are worth placing on record, and they are here presented in the hope that they may add in some measure to the sum total of useful information.

ACKNOWLEDGMENT.

The writers take pleasure in acknowledging the assistance received from the Dean of the Department of Engineering of the University of Michigan in granting the use of the University laboratories and equipment for carrying on these tests, to Gardner S. Williams, M. Am. Soc. C. E., for suggestions which were of assistance in securing accuracy and uniformity in the results, and to H. H. Atwell, Assoc. Am. Soc. C. E., and Messrs. J. Schmutz and L. H. Neilson, of the Faculty of Surveying, and Messrs. H. A. Shuptrine, E. Olmstead, L. R. Manville, and M. D. Bensley, students in Engineering, all of the University of Michigan, who gave their time and work to assist in carrying on the tests.

AIM OF THE TESTS.

The tests were planned to determine the longitudinal and lateral deformations of the columns under various loads, increasing from zero to something above the probable maximum working stress, with varying quantities of reinforcement and at different ages, also the ultimate strength and the load at the first visible sign of failure. From the deformation could be determined the change in unit length for various loads, the modulus of elasticity, and Poisson's ratio. Comparison of the results should show the influence of the reinforcement on these several quantities.

DESCRIPTION OF THE COLUMNS.

All columns were approximately 4 in. in diameter and 27 in. long, over all, cast in a galvanized-iron mould set on a wooden bottom. The mixture throughout was the same, 1:2:4 by loose volume. The cement was Peninsular brand, bought from a dealer from stock, in fair condition, and as ordinarily used in construction. It was sampled and tested as for construction work, passing satisfactorily the standard tests recommended by the Special Committee (of the American Society of Civil Engineers) on Uniform Tests of Cement. The sand was bank sand from a drift deposit at Ann Arbor, secured from a contractor who made a business of furnishing it to the building trades, and was of a grade acceptable for first-class work. The granulometric composition is shown by Table 2. The coarse aggregate was broken limestone from the quarries at Monroe, Mich., and was passed through a $\frac{3}{4}$-in. and retained on a $\frac{1}{4}$-in. screen.

The mixing was done by hand, and under the personal supervision of the writers. In order to fill well about the spirals, it was necessary to use a wet mixture; it was too wet to be tamped with a small rod, but not so wet as to cause the materials to separate in pouring.

The reinforcement in Columns 10 to 14 was furnished by a commercial firm, built up and ready to be placed in the forms. The spiral was of high-carbon steel, $\frac{1}{8}$ in. in diameter, spaced as shown in Table 3. The spacing members were four $\frac{1}{8}$ by $\frac{1}{16}$-in. steel bars. No tests were made to determine the physical constants of this reinforcing material. These columns were tested only for ultimate strength and first sign of failure, and, for purposes of comparison, are included with similar columns tested at other laboratories.

The reinforcement in Columns 15 to 22 was wound on the concrete shaft, after casting, by placing the column in a lathe and winding the wire on under constant tension—sufficient to bed it slightly in the concrete. This was done just before testing, at the age of 28 days. The end of the wire was anchored securely to the column very near one end, the tension was applied, and a dozen turns were wound on as close as they would lie in order to insure against failure at the end; the spiral was then spaced by using the lead screw until within 1 in. of the other end; then another dozen turns were run on close together. The wire was then brought under the anchor and fastened, the tension was released, and the wire cut free. On these columns the wire had a diameter of 0.079 in., an area of 0.0049 sq. in., and the tension, as given by a spring balance, was 62½ lb. on some columns and 125 lb. on others, corresponding to unit stresses of 12 750 and 25 500 lb. per sq. in.

In the remaining columns in which reinforcement was used the spirals were made up and set in place in the form, and the concrete was then poured. The longitudinal members of the reinforcement were barely heavy enough to hold the spiral in place; they were strips of No. 24 gauge galvanized iron, $\frac{7}{16}$ in. wide, with notches, $\frac{7}{32}$ in. deep, accurately spaced by machine; the consecutive turns of the spiral were dropped into these notches and fastened by pinching down the lugs formed in cutting the notches. Four such spacing strips were used for each coil, and, as they were so light, considerable care was necessary to insure the proper location of the reinforcement. On final test to destruction, however, all spirals were found to have been spaced and

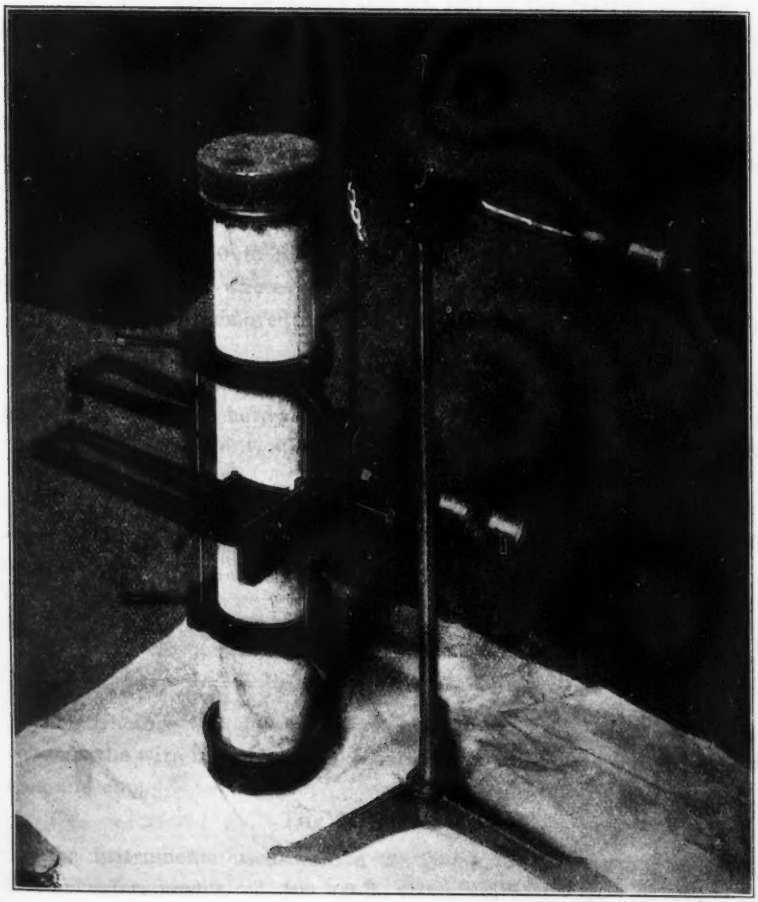

FIG. 1.—EXTENSOMETER.

centered accurately, although in some cases the longitudinal members had taken a spiral position, as may be seen on Fig. 3. A number of extra turns were given at each end of the coil, to make sure that failure should occur in the middle of the shaft, and a circular piece of ¼-in. mesh hardware cloth was placed in each end of the column to give it additional strength. In spite of these precautions, however, a number failed by splitting or crushing at the end, thus failing to develop the full strength of the shaft. The longitudinal spacing strips were not taken into account in estimating the percentage of reinforcement, as they were considered to be too slender to affect sensibly the column under test.

After being cast the columns stood for 24 hours in the forms, and then, except Nos. 10 to 22, were placed in water in the large naval experimental tank, where they remained until removed for testing. Nos. 10 to 22 were removed from the forms at 24 hours and cured in air in the laboratory.

The wire used in the spirals was of unannealed, copper-washed, soft steel, bought from a hardware dealer from stock. Under test it showed an average yield point, ultimate strength, and reduction of area as given in Table 1.

TABLE 1.

Diameter, in inches.	Ultimate strength.	Yield point.	Reduction of area. Percentage.	No. of tests.
0.105	92 330	85 780	50	35
0.079	95 260	87 870	50	45
0.062	78 000	70 000	80	45

The fracture was deeply cupped in all cases, and on crushing the columns the wire invariably broke with the same appearance of reduced area and cup.

The Instruments.

The instruments used were a 200 000-lb. Riehlé testing machine and an extensometer. A 100 000-lb. Olsen machine was available, but the larger clearance, the greater freedom from vibration, and the fact that it was not so frequently in use made the larger machine more desirable. The column was placed in the machine on a cushion of sand in a cast-iron cup which had a machined surface for bearing on the bed of the testing machine.

The extensometer, Fig. 1, was designed by the writers and built in the Instrument maker's shop of the University. The small variation to be measured rendered necessary extreme care and accuracy in construction, and especially the elimination of all lost motion. Knife-edges were ground straight in a Universal grinding machine after tempering, and the V-notches for the knife-edge bearings were made in two parts and similarly ground to true lines. The five rollers of tempered steel were ground to true cylinders, and as nearly of the same size as practicable, and afterward Nos. 2, 3, 4, and 5 were calibrated against No. 1 in a specially designed device.

The small amount of lost motion is shown by the fact that when the column was loaded and the scale-beam rose and fell between its stops a distance of about $\frac{3}{4}$ in. at the free end, the lateral mirrors traveled to and fro on the scale, showing clearly the increase and decrease of diameter of the column due to the small variation of load produced by this motion of the scale-beam.

The construction of the instrument is shown by Figs. 1 and 2. For longitudinal measurement, there are two cast-iron rings, *q*, connected by three telescoping rods, each made up of the rods, *r* and *s*, and the tube, *t*. The rings, *q*, are fastened to the column by setting up the screws, *w*, to a firm bearing. The rods, *r* and *s*, are then set in their seats in the rings and the nuts lightly screwed down on the spherical heads, as shown in the small sectional view. The tube, *t*, is slipped on and firmly clamped to the short rod, *r*, the longer rod, *s*, being left free to slide within *t* as the column shortens under load. The bar, *u*, attached to *t*, holds the roller between its face and the bar, *s*, being held to a steady pressure by the adjustable spring, *v*. The average of the readings of the three rods gives the deformation of the column.

For lateral measurement, the frame, *a*, with its arms attached, is hung by a chain from the counterbalance arm shown in Fig. 1, by the hook at *g*, which is fast to the sliding bar, *e*, which is moved by *h*. The bar, *e*, and the counterweight on *d* are adjusted until the bar, *a*, hangs level, and the weight on the counterbalance arm is adjusted until it exactly balances the weight of the instrument. It is then adjusted to the column in place in the testing machine, with the knife-edges, *o*, and the screws, *b*, bearing against the small blocks of plate glass cemented to the column. The pieces, *j* and *k*, are bolted rigidly to *a* when in use. The arm, *m*, is held to a firm bearing on its

knife-edges, n and o, by a spring and plunger in p. The distance between these knife-edges is 1 in. and the length from n to the end of m, on which the roller bears, is 10 in. The roller is placed between the end of m and the bar, l, the latter being held by a spring to a firm

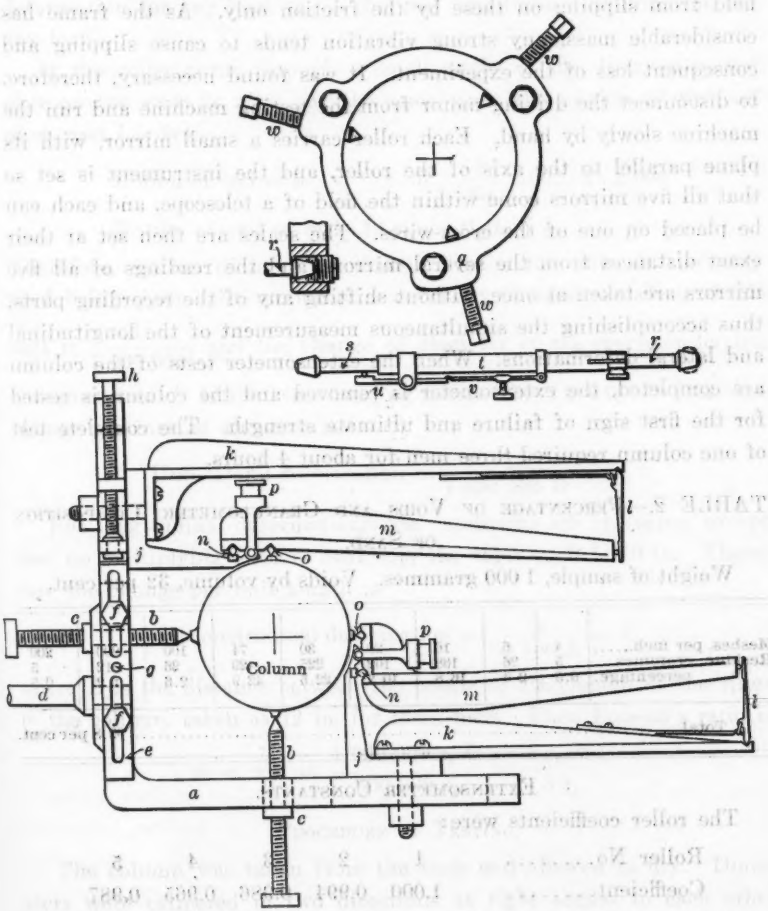

FIG. 2.

bearing against both its knife-edge seat on k and the roller. When all is adjusted, the screws, b, are made fast by the lock-nuts, o, so that all is rigidly connected to a except the arms, m, and the rollers. Any increase or decrease of diameter will cause m to rotate about n

and produce rotation of the roller. The average of the change shown on the two diameters is taken as the change in diameter of the column. The instrument when adjusted is free to move with the column, but none of its weight is carried by its bearing on the glass plates; it is held from slipping on these by the friction only. As the frame has considerable mass, any strong vibration tends to cause slipping and consequent loss of the experiment. It was found necessary, therefore, to disconnect the driving motor from the testing machine and run the machine slowly by hand. Each roller carries a small mirror, with its plane parallel to the axis of the roller, and the instrument is set so that all five mirrors come within the field of a telescope, and each can be placed on one of the cross-wires. The scales are then set at their exact distances from the several mirrors, and the readings of all five mirrors are taken at once, without shifting any of the recording parts, thus accomplishing the simultaneous measurement of the longitudinal and lateral deformations. When the extensometer tests of the column are completed, the extensometer is removed and the column is tested for the first sign of failure and ultimate strength. The complete test of one column required three men for about 4 hours.

TABLE 2.—PERCENTAGE OF VOIDS AND GRANULOMETRIC COMPOSITION OF SAND.

Weight of sample, 1 000 grammes. Voids by volume, 32 per cent.

Meshes, per inch.....	4	6	16	20	30	74	100	200	Passed. 200
Residue, grammes....	5	26	168	102	225	429	26	12	5
" percentage.	0.5	2.6	16.8	10.2	22.5	42.9	2.6	1.2	0.5
Total.....									99.8 per cent.

EXTENSOMETER CONSTANTS.

The roller coefficients were:

Roller No.....	1	2	3	4	5
Coefficient.....	1.000	0.994	0.986	0.965	0.987

Each reading from a roller is multiplied by its coefficient to give the corrected reading.

Referring to Fig. 2: From n to the end of m is 10 times the length n-o. The roller between m and l has a diameter, $2r = \frac{1}{10}$ in., approximately. R is the distance from the center of the roller to the scale.

taken as 100 in. for the transverse extensometer and 50 in. for the longitudinal one. s_1 is the distance traversed on the scale by the lateral mirror when a load is applied, corrected as above stated. s_2 is the same for the longitudinal mirror. λ_1 is the unit lateral deformation for any load. λ_2 is the unit longitudinal deformation for any load.

If the roller turns through a small angle, θ, the distance passed over on the scale is $2R\theta$. At the same time, the motion of the end of m past l is $2r\theta$.

$$\text{Motion of end of } m : s_1 :: r : R :: \frac{1}{20} : 100 :: 1 : 2\,000.$$

Motion of end of m is 10 times that of o.

Therefore, motion of $o : s_1 :: 1 : 20\,000$. Hence the unit on the scale used being $\frac{1}{50}$ in., the motion of o necessary to make a reading of this unit is $\frac{1}{1\,000\,000}$, and the change of diameter of the specimen having a diameter of D is the scale reading divided by 1 000 000. Thus the change per inch of diameter is:

$$\text{Unit lateral deformation} = \frac{s_1}{1\,000\,000\,D} = \lambda_1.$$

For longitudinal deformations, the conditions are the same, except that no multiplying lever is used, and the value of R is 50 in. Therefore the change per unit length is

$$\text{Unit longitudinal deformation} = \frac{s_2}{50\,000\,L} = \lambda_2,$$

where L is the distance between the points of attachment of the rings to the column, taken as 12 in. for these tests. Then Poisson's ratio is

$$m = \frac{\lambda_1}{\lambda_2} = \frac{1\,000\,000\,s_2\,D}{600\,000\,s_1} = \frac{5\,s_2\,D}{3\,s_1}.$$

Procedure in Testing.

The column was taken from the tank and allowed to dry. Diameters were calipered in two directions at right angles to each other at the bottom, center, and top. Two circles with planes normal to the axis were struck, 12 in. apart, to locate the attachment of the rings for the longitudinal measurement. A third was similarly struck for the position of the points for the lateral measurement. On this last circle, the extremities of two diameters at right angles to each

other being located, small blocks of plate glass were cemented to the column with faces made parallel by pressing them to their bed between two parallel steel faces. When these were firmly set, the rings were attached and the column was set with top down on the bed of sand in the cast-iron cup. Cotton waste was tamped lightly around the column to retain the sand. The column, with cap in place, was then set in the testing machine in an upright position, the bottom end on a bed of sand in a cup like that used at the top, and a light load applied to hold it in place, care being taken to secure a full even bearing on the sand. The transverse extensometer was set and adjusted as heretofore described.

The zero reading was taken with a total load of 100 lb. on the column, and thereafter increasing loads were applied and readings taken at each load, up to a maximum, then decreasing loads down to zero were read. On a few columns the loads were run up and back a second time, but usually only one run was made. These readings being completed, the extensometer was removed and the column was tested to destruction.

Behavior Under Test.

The driving gears of the testing machine caused considerable vibration when in action, and in some cases caused the lateral extensometer to slip on the glass plates. The readings showed plainly when this took place, and several tests were necessarily discarded on this account. By running the machine slowly by hand, this trouble was minimized, and usable results were obtained. Usually, the readings were fairly uniform and consistent, but, occasionally, erratic results were found, such as one mirror starting off with negative readings for a few loads, and gradually returning to positive ones as the loads were increased. There was commonly a noticeable difference in the readings between the two lateral, or among the three longitudinal, mirrors, showing that the column underwent lateral deflection. However, when the readings were averaged and plotted, the results usually were very fair curves.

In applying the increasing loads, the weight was set at the proper mark and the load was run up until the beam rose, the reading then being taken. The beam was never held up by the column, but always dropped showing progressive deformation.

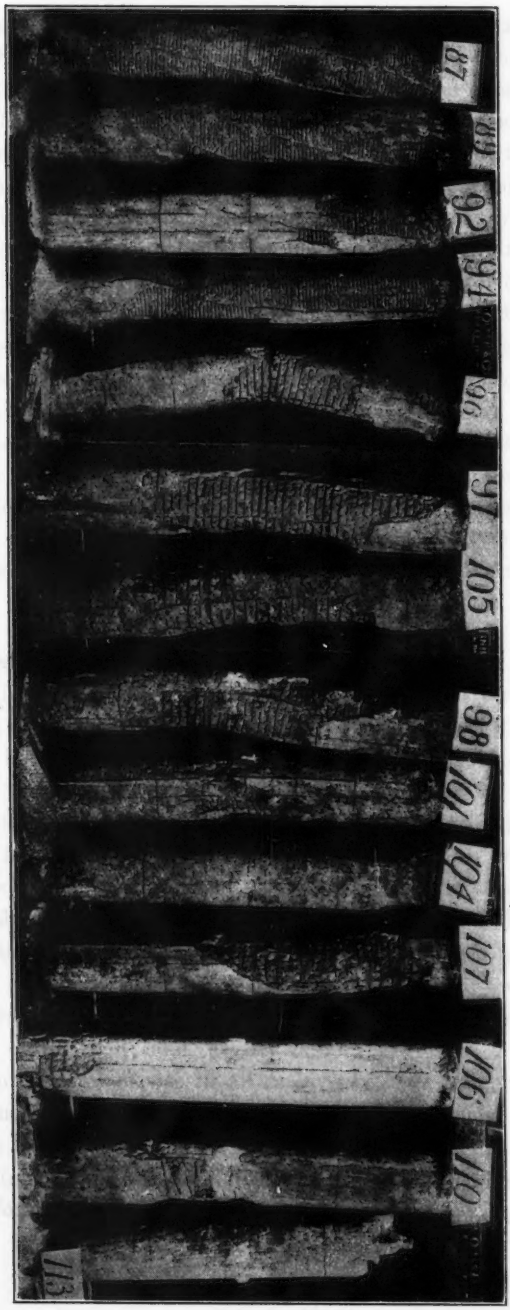

FIG. 3.—APPEARANCE OF CERTAIN TYPICAL COLUMNS AFTER TEST.

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In loading to destruction, considerable lateral deflection usually appeared, the column frequently approximating the characteristic curve of the slender column with fixed ends. Extreme care was taken to observe the first sign of failure, which was shown by a flaking of the surface cement, commonly near the center of length of the column. In the columns without reinforcement, failure was abrupt, only two or three giving any previous indication, and that at a very small percentage below the ultimate load. In those with spiral reinforcement, it was always gradual, giving ample warning and time for observation. The outer concrete separated from the core in considerable sections and fell away, leaving the spiral exposed; with increasing loads, deflection appeared, the concrete of the core began to force out the hooping, and finally the latter broke, the concrete flying out at the point of the break as though blown out by a small charge of explosive; the wire showed a clean tension break. In a few cases the column failed at or near the end, showing imperfect construction. These cases were discarded unless the values were fairly high, as the local failure prevented the column from developing its full strength.

Computation of Values.

The area of cross-section was computed for all plain columns as a circle having a diameter equal to the mean of the six measured, two each at top, center, and bottom. The unit load for values of s_1, s_2, e, and m, for both plain and reinforced columns, are based on the area thus found.

For reinforced columns, the ultimate unit strength and the unit load at the first sign of failure are based on the area of core enclosed by the spiral. The percentage of steel is based on the volume of steel and the volume of core. The percentage shown in Column 14 of Table 3 is the ratio of total load at first sign of failure to total ultimate load. The average lateral deformation is the average of the distances traversed by the lateral extensometer mirrors on the scale, in fiftieths of an inch; and, as the column diameter on which this measurement was taken was in all cases very near 4 in., this reading gives the increase in lateral dimension of the unit cube in units of $\frac{1}{4\,000\,000}$ in. Similarly, the longitudinal deformation is the average of the readings of the three longitudinal mirrors, and gives the longitudinal deformation of the unit cube in units of $\frac{1}{600\,000}$ in. The

value of Poisson's ratio would be approximately $\frac{4\,000\,000\,s_2}{600\,000\,s_1}$, or $\frac{20\,s_2}{3\,s_1}$, but, in computing this, the exact diameter was used instead of the approximation of 4 in., so that the coefficient varies slightly from $\frac{20}{3}$.

The value of $e = \frac{E \text{ of steel}}{E \text{ of concrete}}$, and is used because it is quite as readily computed and plotted as the E of concrete, and, for purposes of this discussion, is somewhat more useful. It is computed as follows:

E of steel is taken as 29 000 000;

f_c = stress on concrete, in pounds per square inch;

$$E_c = \frac{600\,000\,f_c}{s_2};$$

$$\frac{E_s}{E_c} = \frac{29\,000\,000\,s_2}{600\,000\,f_c} = \frac{145\,s_2}{3\,f_c}.$$

The 10-in. slide-rule was used wherever time could be saved thereby, except for values which could be taken directly from tables.

Figs. 4 to 9, inclusive, show typical curves of s_2, e, and m, plotted to uniform scale, for columns Nos. 38, 46, 51, 58, 61, and 63. The unit for the s_2 curves is $\frac{1}{600\,000}$ in., that for e and m is unity. The curves for any column in the series may be plotted from the data for s_2, e_1, and m, in Tables 5, 6, and 7. The curves on Figs. 4 to 9, however, show the effect of unloading the column, as well as loading, but the tables do not.

Comparison of Ultimate Strength.

The effect of the reinforcement on the ultimate strength is shown by the comparisons in Table 4. The columns are grouped as follows:

Group 1.	Reinforcement	0.00 per cent.
2.	"	0.44 to 0.54 per cent.
3.	"	0.67 to 0.86 "
4.	"	1.16 to 1.35 "
5.	"	1.41 to 1.69 "
6.	"	2.02 to 2.32 "
7.	"	4.10 per cent. (One column only.)

TABLE 3.—List of Columns Tested, and Results of Tests of Ultimate Strength.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Column No.	Average diameter.	Mean area.	Age, in days.	Diameter of coil.	Diameter of wire.	Pitch of coil.	Tension.	Percentage.	First Sign.		Ultimate.		Percentage.
									Total.	Unit.	Total.	Unit.	
10	4.03	12.76	28	3.6	0.105	0.50	.....	1.87	18 000	1 770	30 770	3 020	58
11	4.05	12.88	28	3.5	0.104	0.75	.....	1.25	23 000	2 390	30 100	3 130	76
12	4.03	12.76	28	3.5	0.134	0.75	.....	2.09	23 000	2 390	32 300	3 360	71
13	4.03	12.76	28	3.5	0.165	1.00	.....	2.30	26 000	2 700	42 100	4 370	62
14	4.06	12.95	28	3.5	0.165	1.50	.....	1.54	20 000	2 080	32 500	3 380	62
15	4.02	12.69	28	4.02	0.079	0.25	62	1.95	32 000	2 520	52 800	5 490	60
16	4.19	13.79	28	4.19	0.079	0.25	62	1.87	35 000	2 540	45 550	4 740	77
17	4.20	13.85	28	4.20	0.079	0.25	125	1.87	37 000	2 670	51 100	5 310	72
18	4.18	13.72	28	4.18	0.079	0.33	125	1.26	38 000	2 760	47 670	4 950	80
19	4.05	12.88	28	4.05	0.079	0.50	62	0.97	34 000	2 640	39 600	4 120	86
20	4.06	12.95	28	4.06	0.079	0.50	125	0.96	27 700	2 140	34 200	3 560	81
21	4.17	13.66	28	4.17	0.079	0.33	125	0.41	34 000	2 490	49 000	5 090	69
22	4.03	12.76	28	4.03	0.079	0.70	125	0.69	37 000	2 900	40 950	4 250	90
36	4.03	12.76	42	.....	.....	.....	.....	.....	14 120	1 108	.....	.....	100
37	4.05	12.88	41	.....	.....	.....	.....	.....	12 260	950	.....	.....	100
38	4.02	12.69	63	.....	.....	.....	.....	.....	22 660	1 790	.....	.....	100
39	4.05	12.88	40	.....	.....	.....	.....	.....	14 560	1 130	.....	.....	100
41	4.05	12.88	40	.....	.....	.....	.....	.....	10 500	815	11 640	905	90
42	4.03	12.76	58	.....	.....	.....	.....	.....	21 980	1 720	.....	.....	100
43	4.07	13.01	73	.....	.....	.....	.....	.....	16 600	1 275	.....	.....	100
44	4.05	12.88	74	.....	.....	.....	.....	.....	17 840	1 385	.....	.....	100
45	4.03	12.76	268	.....	.....	.....	.....	.....	34 980	2 740	.....	.....	100
46	4.03	12.76	296	.....	.....	.....	.....	.....	37 700	2 955	.....	.....	100
47	4.05	12.88	288	.....	.....	.....	.....	.....	38 000	2 955	.....	.....	100
48	4.04	12.82	414	.....	.....	.....	.....	.....	45 800	3 570	.....	.....	100
49	4.02	12.69	71	.....	.....	.....	.....	.....	29 890	2 345	.....	.....	100
50	4.05	12.88	65	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
50a	4.03	12.76	340	.....	.....	.....	.....	.....	37 000	2 900	.....	.....	100
51	4.04	12.82	434	.....	.....	.....	.....	.....	46 300	3 600	.....	.....	100
52	4.04	12.82	440	.....	.....	.....	.....	.....	35 000	2 725	.....	.....	100
53	4.02	12.69	295	.....	.....	.....	.....	.....	39 570	3 120	.....	.....	100
54	4.05	12.88	441	.....	.....	.....	.....	.....	40 700	3 160	.....	.....	100
56	4.04	12.82	58	3.50	0.079	0.33	.....	1.68	25 000	2 600	40 590	4 220	61
57	4.02	12.69	59	3.56	0.079	0.33	.....	1.65	24 260	2 430	39 000	3 940	62
58	4.03	12.76	61	.....	.....	.....	.....	.....	22 420	1 760	.....	.....	100
59	4.05	12.88	67	3.50	0.105	0.75	.....	1.31	30 600	3 180	40 250	4 184	76
60	4.04	12.82	311	3.62	0.079	0.75	.....	0.72	32 000	3 100	38 200	3 700	83
61	4.04	12.82	77	.....	.....	.....	.....	.....	29 450	2 300	.....	.....	100
62	4.03	12.76	434	.....	.....	.....	.....	.....	36 350	2 850	.....	.....	100
63	4.05	12.88	395	3.69	0.105	0.75	.....	1.24	27 500	2 575	37 500	2 910	73
64	4.03	12.76	435	3.88	0.079	0.75	.....	0.67	37 500	3 180	41 500	3 520	90
65	4.05	12.88	316	3.62	0.105	0.75	.....	1.26	37 500	3 630	42 160	4 085	88
66	4.03	12.76	74	3.62	0.079	0.75	.....	0.72	19 570	1 895	29 000	2 810	67
67	4.02	12.69	323	.....	.....	.....	.....	.....	31 300	2 470	.....	.....	100
68	4.03	12.76	321	3.69	0.105	0.75	.....	1.24	31 400	2 940	36 570	3 420	85
69	4.03	12.76	323	3.62	0.079	0.75	.....	0.72	24 700	2 390	31 350	3 040	78
70	4.04	12.82	393	3.37	0.079	0.25	.....	2.32	27 000	3 020	53 690	6 000	50
71	4.05	12.88	317	3.56	0.062	0.75	.....	0.45	27 120	2 720	27 450	2 770	98
72	4.03	12.76	388	3.62	0.079	0.75	.....	0.72	31 000	3 000	33 500	3 246	92
73	4.04	12.82	428	3.62	0.105	0.75	.....	1.26	46 500	4 500	48 600	4 710	95
74	4.06	12.95	370	3.62	0.062	0.75	.....	0.44	47 000	4 550	47 800	4 630	98
75	4.04	12.82	300	3.56	0.062	0.62	.....	0.54	34 000	3 400	40 500	4 090	83
76	4.03	12.76	376	.....	.....	.....	.....	.....	27 250	2 140	28 000	2 200	97
77	4.03	12.76	277	3.62	0.105	0.62	.....	1.51	38 000	3 680	42 700	4 140	89
78	4.04	12.82	281	3.56	0.062	0.62	.....	0.54	19 000	1 900	31 300	3 160	60
79	4.06	12.95	416	.....	.....	.....	.....	.....	28 060	2 170	.....	.....	100
80	4.05	12.88	417	.....	.....	.....	.....	.....	29 000	2 250	.....	.....	100
81	4.02	12.69	270	3.62	0.079	0.62	.....	0.86	34 000	3 290	36 250	3 520	93
82	4.04	12.82	413	3.62	0.062	0.62	.....	0.52	33 500	3 240	33 700	3 265	99
84	4.03	12.78	42	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

TABLE 3.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Column No.	Average diameter.	Mean area.	Age, in days.	Diameter of coil.	Diameter of wire.	Pitch of coil.	Tension.	Percentage.	First Sign.		Ultimate.		Percentage.
									Total.	Unit.	Total.	Unit.	
85	4.05	12.91	42	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
87	4.03	12.78	35	3.37	0.105	0.25	.....	4.10	27 600	3 090	71 300	7 965	39
89	4.04	12.84	35	3.37	0.079	0.25	.....	2.31	21 600	2 420	49 900	5 580	43
90	4.04	12.80	35	3.37	0.079	0.25	.....	2.31	.....	.....	.....	.....	.....
92	4.04	12.80	34	3.37	0.062	0.25	.....	1.41	21 500	2 410	24 000	2 680	89
93	4.03	12.76	34	3.37	0.062	0.25	.....	1.41	.....	.....	.....	.....	.....
94	4.03	12.76	35	3.37	0.062	0.25	.....	1.41	26 500	2 970	35 270	3 940	75
95	4.03	12.78	35	3.37	0.105	0.50	.....	2.02	.....	.....	.....	.....	.....
96	4.03	12.74	35	3.37	0.105	0.50	.....	2.02	22 600	2 530	42 700	4 770	53
97	4.03	12.76	56	3.37	0.105	0.50	.....	2.02	33 300	3 730	51 580	5 760	64
98	4.04	12.81	31	3.37	0.079	0.50	.....	1.16	19 140	2 140	31 500	3 520	61
99	4.03	12.77	31	3.37	0.079	0.50	.....	1.16	.....	.....	.....	.....	.....
100	4.04	12.80	31	3.37	0.079	0.50	.....	1.16	.....	.....	.....	.....	.....
101	4.03	12.74	29	3.37	0.062	0.50	.....	0.71	16 000	1 790	22 000	2 460	73
102	4.04	12.80	29	3.37	0.062	0.50	.....	0.71	.....	.....	.....	.....	.....
104	4.04	12.79	35	3.37	0.105	0.75	.....	1.35	17 000	1 900	24 500	2 740	69
105	4.05	12.88	49	3.37	0.105	0.75	.....	1.35	27 000	3 020	40 000	4 470	67
106	4.04	12.80	44	3.37	0.105	0.75	.....	1.35	32 600	3 650	34 800	3 890	93
107	4.03	12.74	35	3.37	0.079	0.75	.....	0.78	22 500	2 520	28 000	3 130	80
110	4.03	12.75	35	3.37	0.062	0.75	.....	0.47	21 000	2 350	23 700	2 650	89
111	4.04	12.78	35	3.37	0.062	0.75	.....	0.47	.....	.....	.....	.....	.....
113	4.04	12.79	35	.....	.....	.....	.....	.....	17 200	1 350	.....	.....	100
114	4.04	12.80	35	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
115	4.04	12.80	35	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

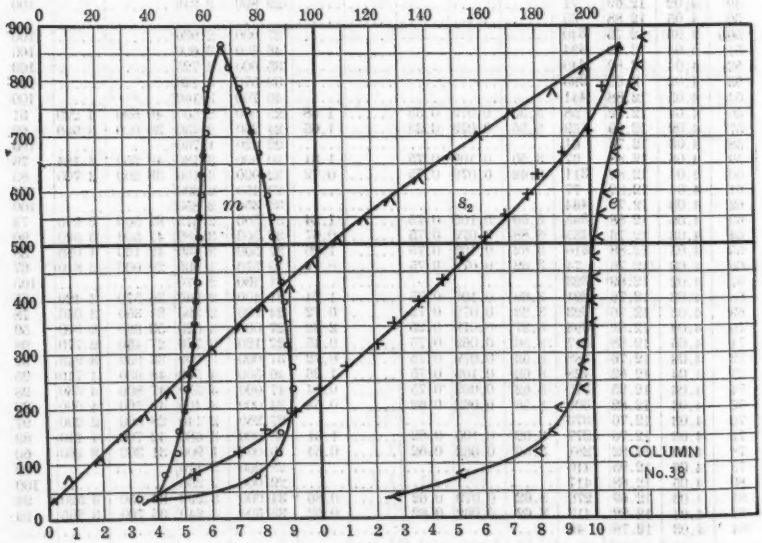

FIG. 4.

Plotting the average ultimate strength of each group, as shown on Fig. 10, gives four points for Group 1 and two for each of the others, except Group 7, which falls entirely off the diagram. The curve of Group 1 shows a consistent increase of strength with age, approximating in form to that of the age-strength curve of neat cement in tension, though the early increase is relatively less. For Groups 2 to 6 the ultimate strength is shown in full lines and the first sign of failure in broken lines.

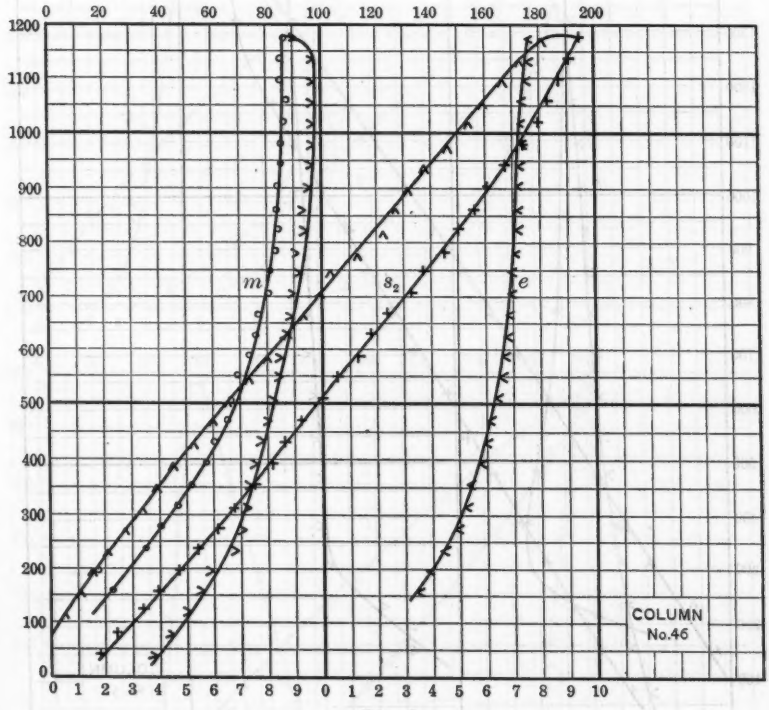

FIG. 5.

There are some irregularities in the relations of these lines, which are probably due to accidental causes, except the reversal of position of 2 and 3, which will be discussed later. However, all groups show an increase of strength with age, and generally with increasing quantities of reinforcement. The load for first sign of failure shows considerable irregularity in the long-time tests in Groups 5 and 6, but, as each of

these sub-groups contains results of one column only, such irregularity is plainly accidental.

Although the relation of the first sign of failure to ultimate strength is brought out more clearly by another diagram, this plotting is given to show the relations within the individual groups. The lack of har-

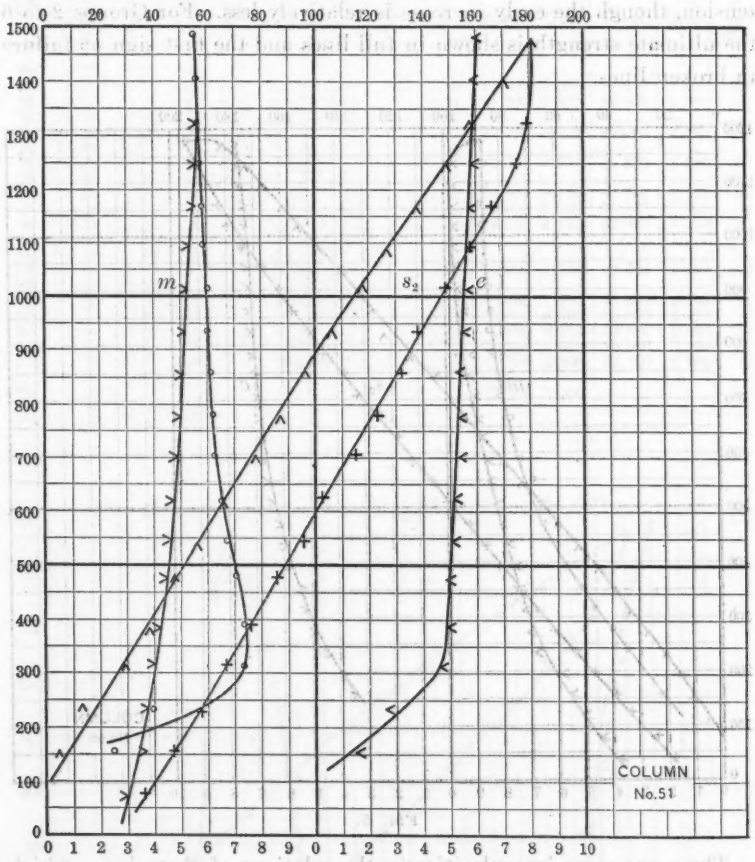

FIG. 6.

mony as the number of tests decreases, and the danger of generalizing from a few tests, are here clearly indicated.

The percentages of Column 14 of Table 3 are plotted on Fig. 11, using percentages of steel as abscissas and the values of Column 14 as ordinates. Here the short-time points, with the exception of that for

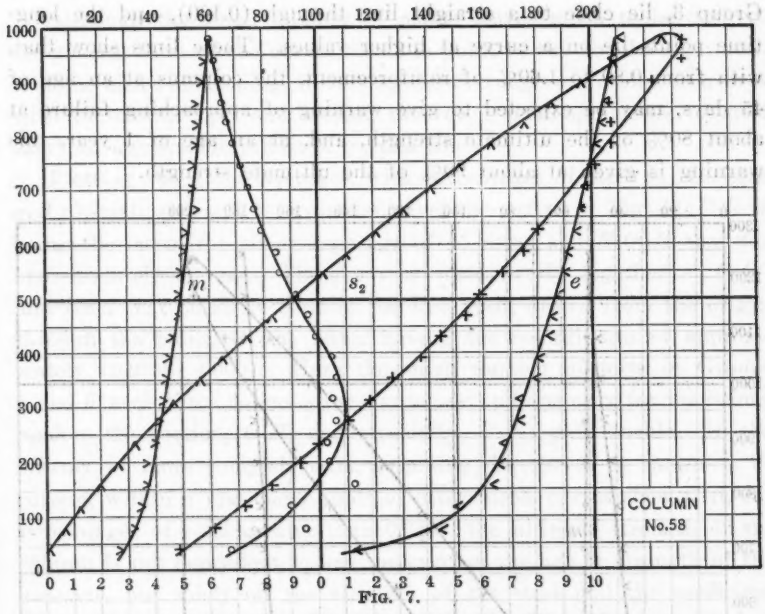
0 20 40 60 80 100 120 140 160 180 200
1000 900 800 700 600 500 400 300 200 100 0
m
s_2
e
COLUMN
No.58
0 1 2 3 4 5 6 7 8 9 0 1 2 3 4 5 6 7 8 9 10

FIG. 7.

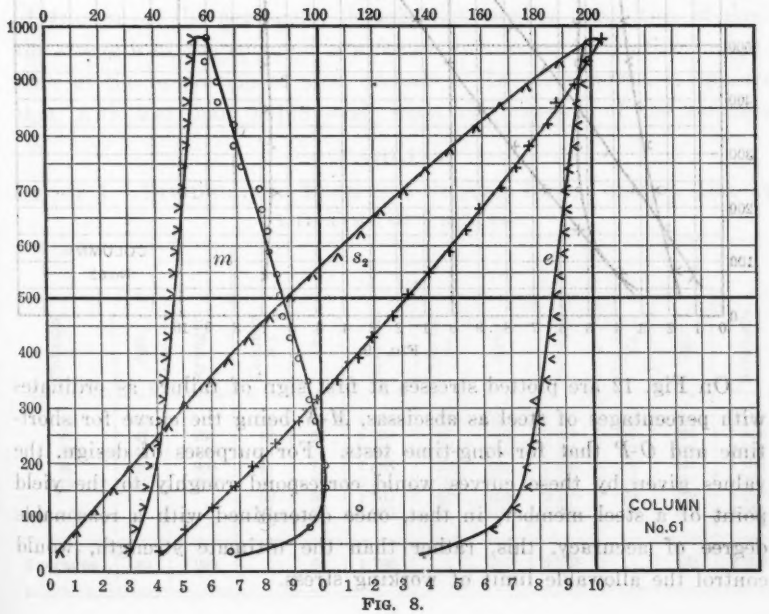
0 20 40 60 80 100 120 140 160 180 200
1000 900 800 700 600 500 400 300 200 100 0
m
s_2
e
COLUMN
No.61
0 1 2 3 4 5 6 7 8 9 0 1 2 3 4 5 6 7 8 9 10

FIG. 8.

Group 3, lie close to a straight line through (0.100), and the long-time points lie on a curve at higher values. These lines show that, with from 0.80 to 1.60% of reinforcement, the columns at an age of 45 days, may be expected to give warning of approaching failure at about 80% of the ultimate strength, and, at an age of 1 year, this warning is given at about 90% of the ultimate strength.

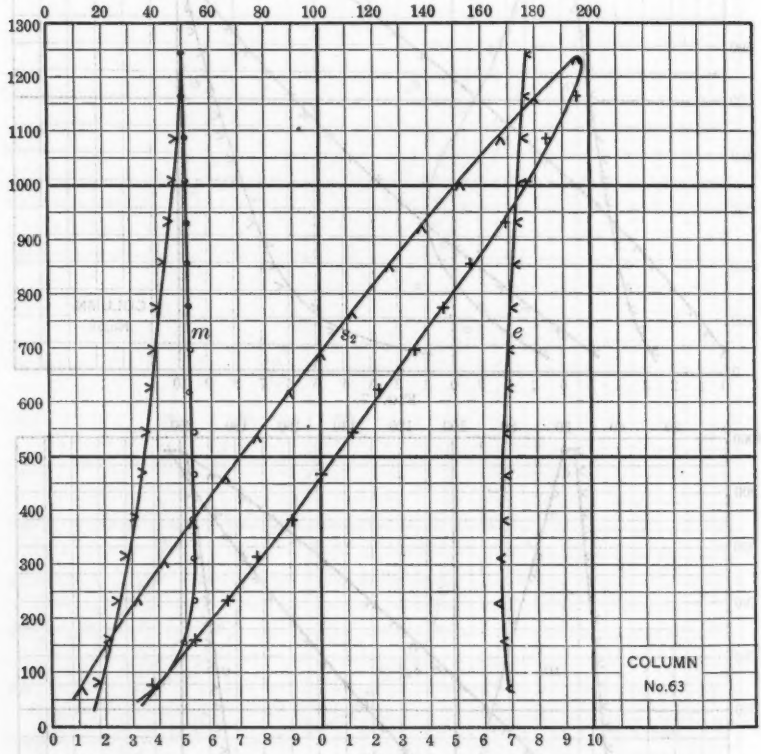

FIG. 9.

On Fig. 12 are plotted stresses at first sign of failure as ordinates with percentages of steel as abscissas, *M-N* being the curve for short-time and *O-P* that for long-time tests. For purposes of design, the values given by these curves would correspond roughly to the yield point of a steel member, in that, once determined with a reasonable degree of accuracy, this, rather than the ultimate strength, would control the allowable limit of working stress.

Although the small number of points leaves much to be desired, the general form of the curves seems to point to two conclusions: first, that the effect of hooping, in raising the first sign of failure, decreases with increasing age; and second, that even small quantities of steel exert an appreciable influence on this limit.

Plotting the group averages, with percentages of steel as abscissas and ultimate strength as ordinates, gives the curves of Fig. 13, *C-G-H* being the curve for an average age of 45 days, and *E-G'-K* that for approximately 1 year. These curves come nearly together at *G-G'*, and from that point lie close to the line, *A-B*, drawn from the origin through the point, *G*, the latter having the co-ordinates of approximately (0.0158, 4 000). Here the small number of tests in Groups 5 and 6, each three columns for short-time and one only for long-time, renders the portion, *G-H*, and especially, *G'-K*, questionable. In the absence of more complete data, assuming the curves as shown to be correct, within a reasonable limit of error, these curves show that, for percentages of steel greater than 0.0158, the ultimate strength of the column is not dependent in any measure on the age or strength of the concrete, but solely on the strength of the steel and the coefficient of friction of the aggregate, though with smaller quantities of steel the strength of the concrete is a factor, increasing in relative importance as the proportion of steel decreases. In other words, it appears that, with less than 0.0158 steel, when the cohesion of the concrete

TABLE 4.

GROUP 1.—DIVIDED INTO FOUR SUB-GROUPS, BASED ON THE AGE AT THE TIME OF TESTING.

(*a*)			(*b*)			(*c*)			(*d*)		
Column No.	Age.	Strength.	Column No.	Age.	Strength.	Column No.	Age.	Strength.	Column No.	Age.	Strength.
36	42	1 110	38	63	1 790	45	268	2 740	48	414	3 570
37	41	950	42	58	1 720	46	296	2 955	51	434	3 600
39	40	1 130	43	73	1 275	47	288	2 955	52	440	2 725
41	40	905	44	74	1 385	50*a*	340	2 900	54	441	3 160
113	35	1 350	49	71	2 345	53	295	3 120	62	434	2 850
			58	61	1 760	67	323	2 470	79	416	2 170
			61	77	2 300	76	376	2 200	80	417	2 250
Averages..	40	1 089	..	68	1 796	..	313	2 763	..	428	2 903

TABLE 4.—(*Continued*).

GROUPS 2 to 6 ARE EACH DIVIDED INTO TWO SUB-GROUPS.

Column No.	Age.	Ultimate strength.	Percentage of steel.	First sign.	Percentage of ultimate.	Column No.	Age.	Ultimate strength.	Percentage of steel.	First sign.	Percentage of ultimate.
GROUP 2.											
110	35	2 650	0.47	2 380	89	71	317	2 773	0.45	2 720	99
						74	370	4 632	0.44	4 550	98
						75	300	4 091	0.54	3 410	84
						78	281	3 160	0.54	1 910	61
						82	413	3 265	0.52	3 240	99
Averages..	..	....	....	....	..	..	336	3 584	0.50	3 166	88
GROUP 3.											
66	74	2 810	0.72	1 890	67	60	311	3 702	0.72	3 100	84
101	29	2 460	0.71	1 790	73	64	435	3 520	0.67	3 170	90
107	35	3 130	0.78	2 520	80	69	323	3 040	0.72	2 390	70
						72	388	3 246	0.72	3 000	93
						81	270	3 520	0.86	3 300	94
Averages..	46	2 800	0.74	2 067	72	..	345	3 406	0.74	2 992	88
GROUP 4.											
59	67	4 184	1.31	3 180	76	63	395	3 520	1.24	2 570	73
98	31	3 520	1.16	2 130	61	65	316	4 085	1.26	3 640	89
104	35	2 740	1.35	1 900	69	68	321	3 420	1.24	2 940	86
105	49	4 470	1.35	3 020	67	73	428	4 710	1.26	4 500	96
106	44	3 890	1.35	3 640	93						
Averages..	45	3 761	1.30	2 774	73	..	365	3 934	1.25	3 412	86
GROUP 5.											
56	58	4 219	1.68	2 600	62	77	277	4 138	1.51	3 680	89
57	59	3 989	1.65	2 430	62						
94	35	3 940	1.41	2 960	75						
Averages..	51	4 033	1.58	2 663	66						
GROUP 6.											
89	35	5 580	2.31	2 410	43	70	393	6 000	2.32	3 020	50
96	35	4 770	2.02	2 520	53						
97	56	5 760	2.02	3 720	64						
Averages..	42	5 370	2.11	2 883	53						
GROUP 7.											
87	35	7 965	4.10	3 080	39						

disappears, the steel does not suffice to carry the load by its lateral hold on the granular, non-coherent mass, but that with the failure of cohesion of the concrete occurs the failure of the column; and, with a larger quantity of steel than this, the failure of cohesion in the

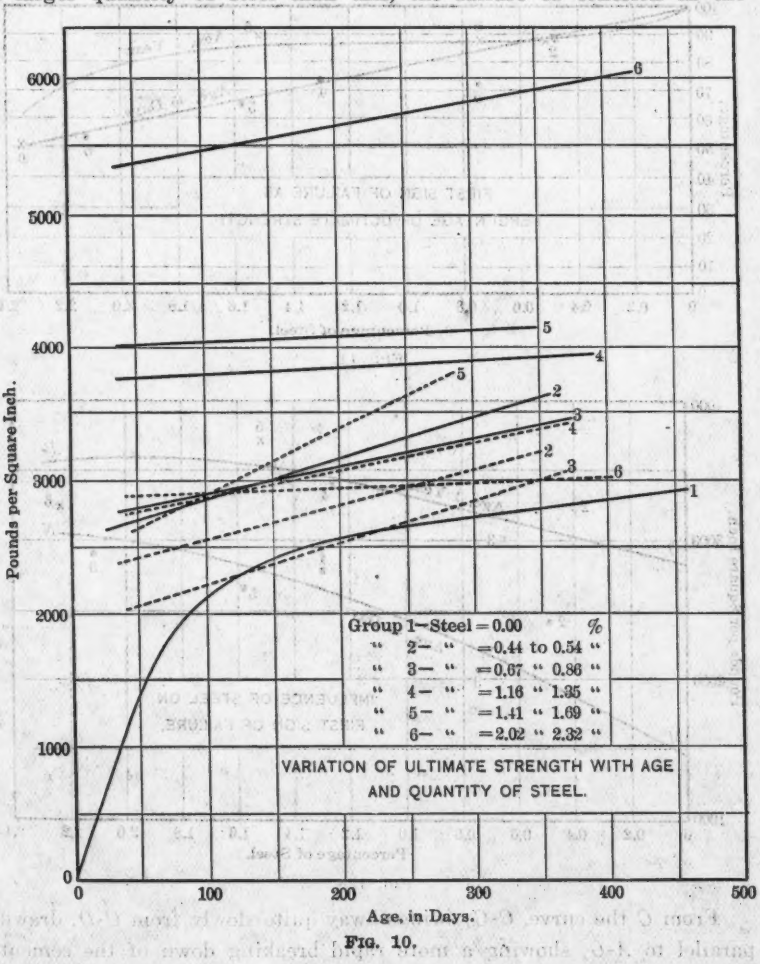

FIG. 10.

concrete does not produce failure of the column, as the steel acting on the core suffices to support the load.

Attention is here directed to the fact that this limit of 0.0158 is based on the volume ratio of steel to concrete, on wire having a yield

point and ultimate strength as shown in the table, and on a broken-stone concrete. With a weaker steel, or with aggregate material having a different coefficient of friction, this limit would be changed.

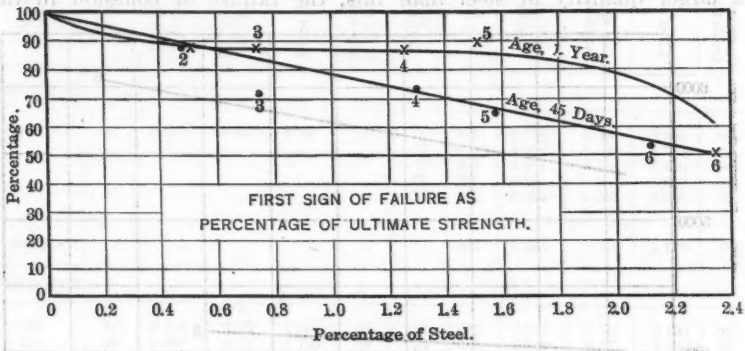

FIG. 11.

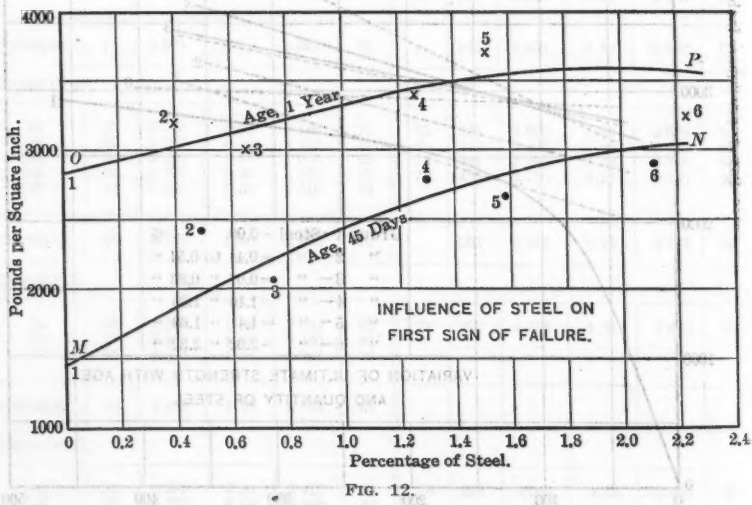

FIG. 12.

From *C* the curve, *C-G*, swings away quite slowly from *C-D*, drawn parallel to *A-G*, showing a more rapid breaking down of the cement bond as the load increases; though *E-G* draws away from the similar parallel, *E-F*, much more rapidly. Both, however, indicate clearly that the reinforcement is effective in raising the ultimate strength, even with small proportions of steel.

The ordinate to the curve at any point consists of two portions: that below *A-B*, due to the wire confining the granular mass, and that above *A-B*, due to the cohesion of the concrete; or, perhaps better, that below *A-B*, due to the reinforcing steel, and that above due to the cement bond.

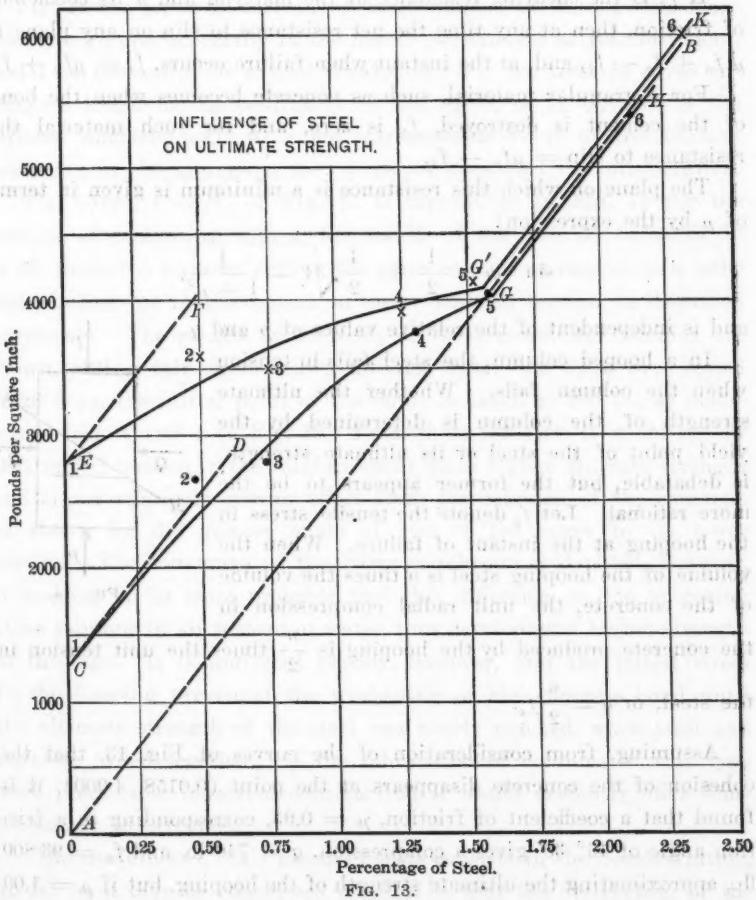

FIG. 13.

By rational methods the following is found, assuming that the column fails by shear on an oblique plane: Given a particle acted on by the forces, P and Q, at right angles to each other, producing stresses p and q on their normal planes (p being greater than q),

the resultant normal and tangential stresses on a plane, M-N, making an angle, ϕ, with the direction of Q will be

$$f_n = p \cos.^2 \phi + q \sin.^2 \phi$$
$$f_t = (p - q) \cos. \phi \sin. \phi$$

If f_z *is* the shearing resistance of the material and μ its coefficient of friction, then at any time the net resistance to slip on any plane is $\mu f_n + f_z - f_t$, and, at the instant when failure occurs, $f_t = \mu f_n + f_z$.

For a granular material, such as concrete becomes when the bond of the cement is destroyed, f_z is zero, and for such material the resistance to slip $= \mu f_n - f_t$.

The plane on which this resistance is a minimum is given in terms of μ by the expression:

$$\sin.^2 \phi = \frac{1}{2} \pm \frac{1}{2}\sqrt{1 - \frac{1}{1 + \mu^2}},$$

and is independent of the relative values of p and q.

In a hooped column, the steel fails in tension when the column fails. Whether the ultimate strength of the column is determined by the yield point of the steel or its ultimate strength, is debatable, but the former appears to be the more rational. Let f_s denote the tensile stress in the hooping at the instant of failure. When the volume of the hooping steel is n times the volume of the concrete, the unit radial compression in the concrete produced by the hooping is $\frac{n}{2}$ times the unit tension in the steel, or $q = \frac{n}{2} f_s$.

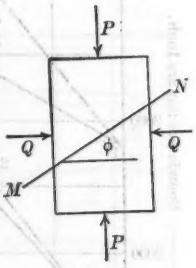

FIG. 14.

Assuming, from consideration of the curves of Fig. 13, that the cohesion of the concrete disappears at the point (0.0158, 4 000), it is found that a coefficient of friction, $\mu = 0.95$, corresponding to a friction angle of 43° 30′ gives a compression, $q = 740$ lb. and $f_s = 93\,800$ lb., approximating the ultimate strength of the hooping, but if $\mu = 1.00$ and the friction angle is 45°, $q = 680$ lb. and $f_s = 86\,000$ lb., or about the yield point of the spiral. It would seem, therefore, that, with a steel having a relatively high yield point, the question whether ultimate strength or yield point determines the strength of the column

is not an important one, because so small a variation in the angle of friction corresponds to so large a range of stress in the steel.

This method of treatment takes no account of flexure in the column, and, therefore, is applicable only to short columns.

The writers do not consider that the tests prove that the curves of Fig. 13 are typically correct. A large number of tests, giving more complete groups, especially in the higher percentages of reinforcement, would be necessary for a final conclusion; but the tests as given indicate a break in the continuity of the curve, as at *G*, and, as the rational analysis leads to a not unreasonable value of the friction coefficient in the aggregate, the curves are presented for consideration.

The curve, *C-G-H*, of Fig. 13 is reproduced in Fig. 15 for the purpose of comparing with it the results of the tests on Columns 15 to 22, inclusive, built by setting the plain column on centers in a lathe and winding the reinforcement on under constant tension, as described previously. The results of the tests of these individual columns are shown, with a fair curve drawn through them. As these were tested at 28 days, the initial point of the curve is taken at 900 lb., as determined from Curve 1 on Fig. 10. From these few tests it would appear that initial tension in the reinforcement gives higher ultimate strength and also a relatively greater effect from the cohesion of the concrete, as shown by the greater angle of rise of the curve in its lower portion. The soundness of the latter conclusion is doubted, however, it seeming to be more probable that this difference is due to curing these columns in air instead of water, thus developing a higher strength at this age. It is indicated plainly, however, that the initial stress in the hooping prevented the weakening of the concrete bond until the ultimate strength of the steel was nearly reached, when steel and concrete failed together. If the fabrication of columns of this type were practicable in actual construction it would evidently offer some advantage in strength.

The usefulness of this group of tests consists in the demonstration, so far as it may be considered conclusive, that the destruction of the bond of the concrete itself is restrained by the hooping, so that this destruction becomes a gradual process, instead of the abrupt one which occurs in the plain column, and that this restraining effect is produced, even with small percentages of steel.

On Fig. 15 are plotted also the points for Columns 10 to 14, inclusive, marked z; also from tests* by A. N. Talbot, M. Am. Soc. C. E., Columns 171, 172, 173, 181, 182, 183, reinforced with high-carbon steel, and marked T, and Columns 176, 177, 178, 186, 187, reinforced

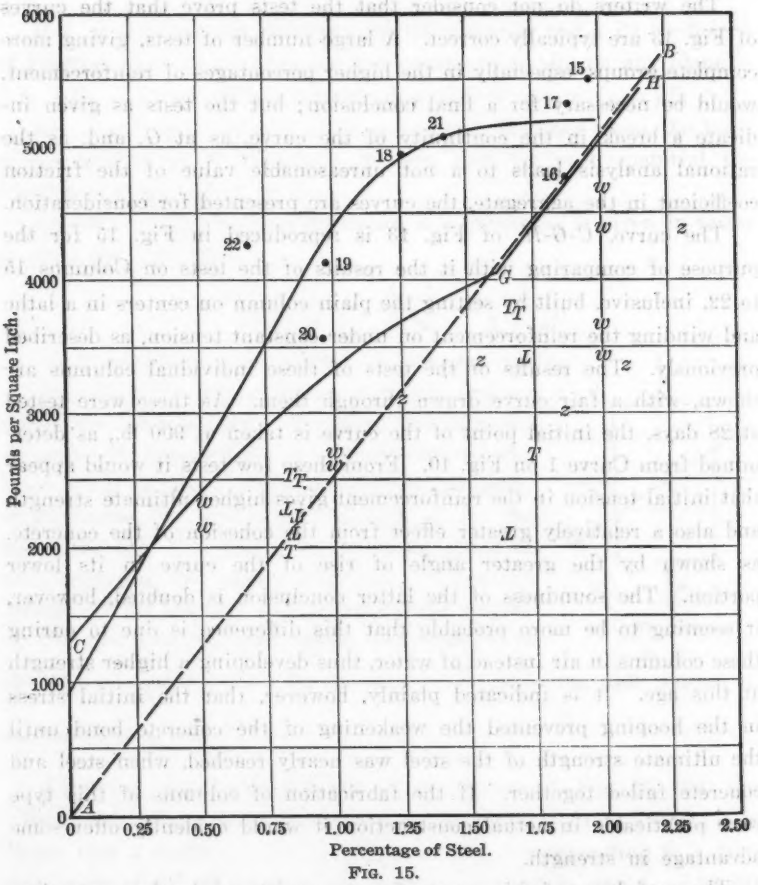

FIG. 15.

with mild steel and marked L; also from tests† by Professor M. O. Withey, Columns $C1$, $C2$, $C3$, $C4$, $H1$, $H2$, $L1$, $L2$, marked w. All the columns of this group are reinforced with hooping prepared by commercial firms, and differ only in minor details. Compared with

* Bulletin No. 20, Illinois Experiment Station.
† Bulletin No. 466, University of Wisconsin.

Columns 15 to 115 of this series, they differ first in having longitudinal or spacing members relatively much heavier, and second in the use of a heavier hooping wire spaced on a higher pitch.

The fact, noted by Considère, that a low-pitch spiral is more effective than one of high pitch is brought out clearly here. These points lie in a fairly close group, but all are well below the curve, *C-G-H*.

There is no reason to believe that these lower values are due to inferior concrete, nor, on account of their consistent grouping, can it be argued that they are due to differences in conditions of the tests, unless, because of the fact that these were cured in air and the *C-G-H* columns were cured in water. A consideration of Fig. 10, however, shows the line of Group 2, with 0.44 to 0.54% of steel, lying above that of Group 3, with 0.67 to 0.86 per cent. It happens that all the columns of Group 2 were reinforced with the 0.061 wire, and that Groups 3, 4, and 5, have each only one column with this fine wire, all other reinforcement being heavier. The writers, therefore, believe that the points under discussion lie below *C-G-H*, because of the fact that the spiral is of coarser wire and consequently higher pitch.

Values of s_2.—The stress-strain curves show a permanent deformation which is greater in the short-time tests. This is shown by arranging in two groups, the first including all columns from 84 to 115, all of which were loaded under the extensometer to 700 lb. per sq. in., and all at approximately the same age. These show on the 100-lb. line a length of 53 units (of $\frac{1}{6000}$ in.) between the ascending and descending curves. The second group includes Columns 46, 48, 51, 52, 53, 54, 60, 62, 64, 67, 69, 71, 73, 74, 75, 79, 80, 81, and 82, all but four of which were loaded to 1 400 lb., and all above 1 150 lb. have an average age of 352 days and show on the 100-lb. line a length of 35 units.

In the majority of cases these curves show a satisfactory uniformity, but some exceptions appear, as Column 72, tested at 388 days, which shows an abnormal lack of resilience, the load dropping from 1 400 to 1 000 lb. before recovery began.

For examining these results to determine the influence of the hooping on them, the columns are grouped in Table 5 in the same manner as for ultimate strength. The strains here are read off from the plotted curves, instead of being taken from the tables.

The average values of these strains, using ages in days as ordinates and strains as abscissas, are plotted on the diagram, Fig. 16, where

the numerals show the group numbers. Separate curves are shown for stresses of 400 to 900 lb. per sq. in. Here appears the unexpected condition that the plain columns show less strain than those which are reinforced. Before beginning the tests the writers had expected that, at least in the higher stresses, the columns would show, both in the s_2 and the m, just the opposite. If the strain in the reinforced columns had been less than in the plain ones, or if no difference between the two had appeared, the explanation would be simple; but, that the presence of a spiral of steel should be accompanied by a greater deformation

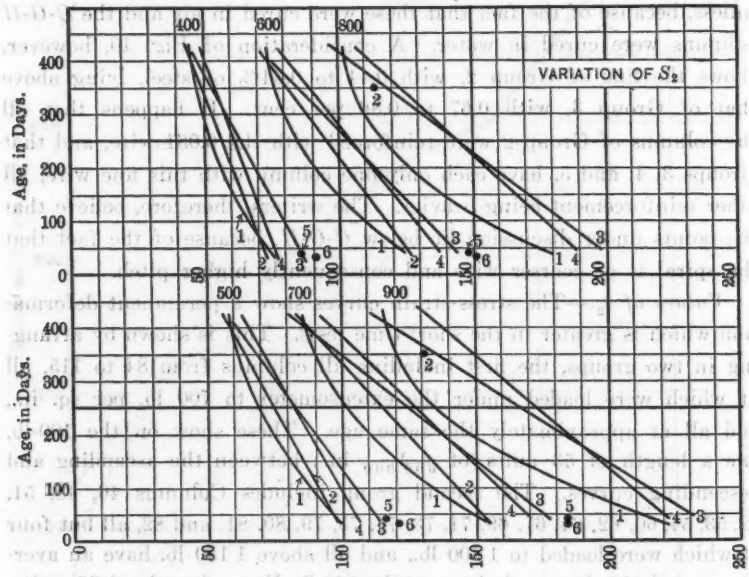

FIG. 16.

does not find a ready reason, unless it be that accidental peculiarities have existed which have escaped observation, but which, nevertheless, were sufficient to produce the condition shown. The consistency with which this runs through all groups leads to the opinion that the same results would be found by a repetition of the tests. That the shrinkage of the concrete, causing initial compression in the steel, might be the cause is untenable, first, because the concrete was immersed in water during the entire aging period, and the shrinkage, therefore, would not occur so as to produce this result; and second, because if the steel spiral were under initial compression one would expect to find

TABLE 5.—VALUES OF s_2.

GROUP 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.....	42	79	108	135	163	.....	.....	.....	.....	.....	.....	.....
114.....	35	68	91	115	144	.....	.....	.....	.....	.....	.....	.....
115.....	35	65	90	115	142	.....	.....	.....	.....	.....	.....	.....
Average....	37	71	96	122	150	.....	.....	.....	.....	.....	.....	.....
50.....	65	70	91	113	136	161	.....	.....	.....	.....	.....	.....
38.....	63	81	104	128	158	188	.....	.....	.....	.....	.....	.....
42.....	58	68	90	113	135	160	.....	.....	.....	.....	.....	.....
43.....	73	85	112	143	176	213	251	.....	.....	.....	.....	.....
44.....	74	78	103	131	160	192	227	.....	.....	.....	.....	.....
58.....	61	67	90	114	140	168	197	.....	.....	.....	.....	.....
61.....	77	68	88	108	130	153	177	.....	.....	.....	.....	.....
Average....	67	74	97	121	148	176	213	.....	.....	.....	.....	.....
46.....	296	47	65	83	98	114	132	149	167	.....	.....	.....
47.....	288	72	89	106	122	138	153	.....	.....	.....	.....	.....
53.....	295	42	54	66	79	92	105	119	132	.....	.....	.....
67.....	322	40	53	67	82	97	112	128	143	160	177	196
Average....	300	50	65	80	95	110	125	132	147	160	177	196
48.....	414	36	48	59	71	84	97	110	123	136	148	161
51.....	434	38	51	63	75	88	100	114	128	142	156	170
52.....	440	44	57	71	85	99	113	128	142	156	170	184
54.....	441	43	54	66	78	90	101	114	128	142	156	170
62.....	434	53	68	82	96	112	128	145	161	178	194	212
79.....	416	45	60	75	91	107	123	140	158	178	198	.....
80.....	417	59	75	94	112	130	150	170	192	215	240	264
Average....	428	45	59	73	86	101	116	132	146	164	180	193

GROUP 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
111.....	35	0.50	73	99	127	155	.....	.....	.....	.....	.....	.....	.....
71.....	317	0.45	51	64	78	92	106	121	137	154	172	190	211
74.....	370	0.40	62	80	99	119	138	157	175	192	210	227	.....
75.....	300	0.54	38	54	70	86	103	120	139	159	181	205	230
82.....	413	0.58	48	61	76	91	108	125	143	162	182	202	223
Average.	350	0.49	50	65	81	97	114	131	148	167	186	206	221

TABLE 5.—VALUES OF s_2.—(*Continued*).

GROUP 3.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
66....	74	0.72	83	107	133	161	191	225	.....	.....	.....	.....	.....
101...	29	0.71	89	120	153	190	.....	.....	.....	.....	.....	.....	.....
107....	35	0.78	83	113	147	179	.....	.....	.....	.....	.....	.....	.....
Average.	46	0.74	85	113	144	177	191	225	.....	.....	.....	.....	.....
60....	311	0.72	66	81	96	111	127	144	162	181	200	220	241
64....	435	0.67	44	59	74	89	105	122	140	158	177	198	218
69....	323	0.72	53	70	86	104	123	143	164	186	208	233	260
72....	388	0.72	57	73	89	105	123	141	160	181	205	230	259
81....	270	0.86	48	66	83	100	120	140	160	181	.....	.....	.....
Average.	345	0.74	54	70	86	102	120	138	157	177	197	220	244

GROUP 4.

59....	67	1.31	76	101	128	157	186	219	256	.....	.....	.....	.....
99....	31	1.16	76	110	146	183	.....	.....	.....	.....	.....	.....	.....
Average.	49	1.23	76	105	137	170	186	219	256	.....	.....	.....	.....
63....	395	1.24	55	70	86	101	117	133	150	169	188	.....	.....
73....	428	1.26	46	60	74	89	104	120	135	152	169	188	206
Average.	411	1.25	50	65	80	95	110	126	142	160	178	188	206

GROUP 5.

56....	58	1.69	84	115	148	184	227	.....	.....	.....	.....	.....	.....
92....	34	1.41	90	121	156	193	.....	.....	.....	.....	.....	.....	.....
93....	34	1.41	89	120	153	187	.....	.....	.....	.....	.....	.....	.....
94....	35	1.41	83	112	142	175	.....	.....	.....	.....	.....	.....	.....
105....	35	1.35	91	125	161	202	.....	.....	.....	.....	.....	.....	.....
106....	35	1.35	77	104	133	164	.....	.....	.....	.....	.....	.....	.....
Average.	38	1.44	86	116	149	184	227	.....	.....	.....	.....	.....	.....

GROUP 6.

89....	35	2.30	90	118	144	172	.....	.....	.....	.....	.....	.....	.....
95....	35	2.04	94	124	160	199	.....	.....	.....	.....	.....	.....	.....
96....	35	2.02	88	120	153	180	.....	.....	.....	.....	.....	.....	.....
Average.	35	2.12	91	121	152	184	.....	.....	.....	.....	.....	.....	.....

m, at short time, less for reinforced than for plain columns, whereas the curves for m show precisely the opposite. The only explanation which the writers can suggest is that the presence of the reinforcement, with its rather closely spaced wires, may have prevented in a measure the thorough elimination of bubbles and small voids, thus rendering the concrete as a whole less dense. Moreover, Professor Talbot, has discussed* the same condition, which he found to exist in his experiments, and on which he places the same interpretation.

These curves indicate that, within the limits of ordinary working stresses, spiral reinforcement does not noticeably lend assistance to the concrete in carrying the load, but that, if such assistance is developed, it is at stresses well above the ordinary working limits.

Values of e.—The values of e have been computed and plotted for the ascending curves only. In Table 6 the columns are grouped as before, and the values are as read from the plotted curves.

The average values from Table 6 are plotted on Fig. 17, the ages, in days, being the ordinates and the values of e the abscissas. The curves show a close agreement for both plain and reinforced columns at 425 days, and separate more widely with decreasing age and increasing stress. As these are derived directly from s_2, they show the same general relations between the plain and reinforced columns, and the comments made regarding the stress-strain curves apply with equal force here. The evidence of these curves is decidedly against the use of values of e greater than 10 for computing the strength of reinforced columns. It runs above that value only for the combinations of short time with high stresses, a combination to be avoided in practice as much as possible.

Values of m.—m is the ratio of longitudinal to lateral deformation, the two being opposite in kind when the member is stressed in one direction only, and its value, m, in terms of the observed quantities, becomes

$$m = \frac{\lambda_2}{\lambda_1} = \frac{5\ D\ s_2}{3\ s_1}.$$

This involves one more observed quantity, s_1, which is more difficult to measure, not only because it is very small, but also because of the likelihood of the instrument slipping. The curves show less

* Bulletin No. 20, Illinois Engineering Experiment Station.

TABLE 6.—VALUES OF e.

GROUP 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.....	42	9.4	10.2	10.8	11.2	.....	.....	.....	.....	.....	.....	.....
114.....	35	8.3	8.9	9.4	9.8	.....	.....	.....	.....	.....	.....	.....
115.....	35	7.8	8.6	9.3	9.9	.....	.....	.....	.....	.....	.....	.....
Average....	37	8.5	9.2	9.8	10.3	.....	.....	.....	.....	.....	.....	.....
38.....	63	9.8	10.1	10.4	10.8	11.5	.....	.....	.....	.....	.....	.....
42.....	58	8.3	8.8	9.2	9.4	9.6	.....	.....	.....	.....	.....	.....
43.....	73	10.4	10.9	11.4	12.0	12.7	13.3	.....	.....	.....	.....	.....
44.....	74	9.6	10.1	10.5	11.0	11.5	12.2	.....	.....	.....	.....	.....
50.....	65	8.4	8.8	9.1	9.4	9.8	.....	.....	.....	.....	.....	.....
58.....	61	8.0	8.6	9.2	9.7	10.2	10.7	.....	.....	.....	.....	.....
61.....	77	8.2	8.5	8.8	9.0	9.3	9.6	.....	.....	.....	.....	.....
Average....	67	8.9	9.4	9.8	10.2	10.7	11.4	.....	.....	.....	.....	.....
46.....	296	5.7	6.2	6.6	6.9	7.0	7.1	7.2	7.4	.....	.....	.....
47.....	288	8.7	8.6	8.5	8.4	8.3	8.2	.....	.....	.....	.....	.....
53.....	295	5.1	5.2	5.3	5.5	5.6	5.7	5.8	5.9	.....	.....	.....
67.....	323	4.9	5.1	5.3	5.5	5.7	5.9	6.1	6.3	6.4	6.5	6.6
Average....	300	6.1	6.3	6.4	6.6	6.7	6.7	6.4	6.5	6.4	6.5	6.6
48.....	414	4.3	4.5	4.8	5.0	5.1	5.2	5.4	5.5	5.5	5.5	5.5
51.....	434	4.9	5.0	5.1	5.2	5.3	5.4	5.5	5.7	5.8	5.9	6.0
52.....	440	5.4	5.7	5.8	5.9	6.0	6.0	6.1	6.1	6.2	6.2	6.3
54.....	441	5.0	5.1	5.2	5.3	5.4	5.5	5.5	5.6	5.7	5.8	5.9
62.....	434	6.4	6.5	6.6	6.7	6.8	6.9	7.0	7.1	7.1	7.2	7.3
79.....	416	5.4	5.7	6.0	6.2	6.4	6.6	6.8	7.0	7.2	7.4	7.5
80.....	417	7.0	7.2	7.4	7.6	7.8	8.0	8.2	8.4	8.6	8.8	9.0
Average....	428	5.5	5.7	5.8	6.0	6.1	6.2	6.4	6.5	6.6	6.7	6.8

GROUP 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100
111.....	35	0.47	8.9	9.6	10.2	10.8	.....	.....	.....	.....
71.....	317	0.45	6.1	6.2	6.3	6.4	6.5	6.5	6.6	6.7
74.....	370	0.44	7.7	7.9	8.1	8.2	8.3	8.3	8.3	8.3
75.....	300	0.54	4.6	5.1	5.5	5.9	6.2	6.4	6.7	7.0
82.....	413	0.52	5.8	6.0	6.2	6.4	6.6	6.8	7.0	7.1
Average.....	350	0.47	6.1	6.3	6.5	6.7	6.9	7.0	7.1	7.3

TABLE 6.—(*Continued*).

GROUP 3.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100
66.....	74	0.72	10.1	10.4	10.7	11.0	11.4	12.0	.....	.....
101.....	29	0.71	10.7	11.5	12.3	13.0	.....	.....	.....	.....
107.....	35	0.78	10.1	11.0	11.8	12.2	.....	.....	.....	.....
Average.....	46	0.74	10.3	11.0	11.6	12.1	11.4	12.0	.....	.....
60.....	311	0.72	8.1	7.9	7.8	7.8	7.8	7.8	7.9	8.0
64.....	435	0.67	5.2	5.6	5.9	6.1	6.3	6.5	6.7	7.0
69.....	323	0.72	6.5	6.9	7.1	7.3	7.4	7.7	8.0	8.2
72.....	388	0.80	6.8	7.0	7.1	7.3	7.4	7.6	7.8	8.0
81.....	270	0.86	5.7	6.1	6.6	6.9	7.2	7.4	7.6	7.8
Average.....	344	0.74	6.5	6.7	6.9	7.1	7.2	7.4	7.6	7.8

GROUP 4.

59.....	67	1.31	9.2	9.8	10.4	10.9	11.3	11.8	12.2	.....
99.....	31	1.16	9.1	10.5	11.8	12.8	.....	.....	.....	.....
Average.....	49	1.23	9.1	10.1	11.1	11.8	11.3	11.8	12.2	.....
63.....	395	1.24	6.6	6.7	6.8	6.9	7.0	7.1	7.3	7.4
73.....	428	1.26	5.5	5.8	6.0	6.2	6.3	6.5	6.6	6.7
Average.....	411	1.25	6.0	6.1	6.4	6.5	6.6	6.8	6.9	7.0

GROUP 5.

56.....	58	1.69	10.1	10.9	11.8	12.7	13.6	14.7	15.9	.....
92.....	34	1.41	10.6	11.7	12.5	13.3	.....	.....	.....	.....
93.....	34	1.41	10.8	11.5	12.2	12.9	.....	.....	.....	.....
94.....	35	1.41	10.0	10.8	11.4	12.1	.....	.....	.....	.....
105.....	35	1.35	10.9	12.0	13.0	14.0	.....	.....	.....	.....
106.....	35	1.35	9.5	10.2	10.8	11.3	.....	.....	.....	.....
Average.....	38	1.44	10.3	11.2	11.9	12.7	13.6	14.7	15.9	.....

GROUP 6.

89.....	35	2.30	10.9	11.4	11.7	11.9	.....	.....	.....	.....
95.....	35	2.04	11.3	12.0	12.8	13.8	.....	.....	.....	.....
96.....	35	2.02	10.5	11.5	12.4	13.1	.....	.....	.....	.....
Average.....	35	2.12	10.9	11.6	12.3	12.9	.....	.....	.....	.....

uniformity than those of s_2 and e, and a few have been thrown out of the general averages where they appeared to differ widely and erratically from the others of the group. In Table 7 the columns are grouped as before.

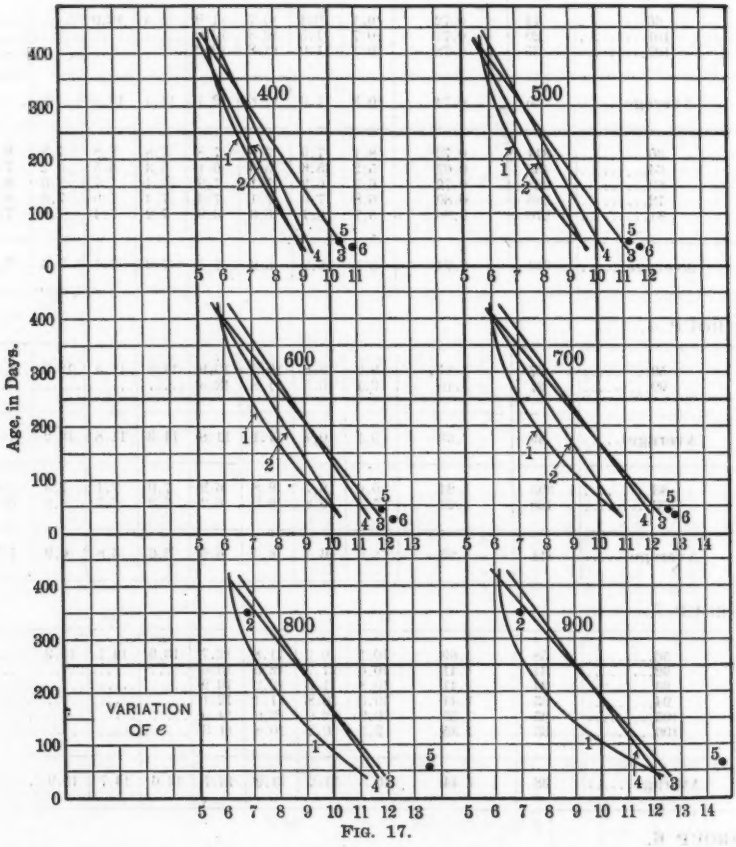

FIG. 17.

The average values from Table 7 are plotted on Fig. 18; with ages, in days, as ordinates, and values of m as abscissas. The single column of Group 2, tested at short time, gives values which for the lower stresses appear to be too high, but the other points lie fairly close together, the value 7 for long time and 8 for short time representing fair averages. On the 800 and 900-lb. curves the value for long time drops below 7. In the few cases where this constant has been used in

discussions, as noted by the writers, it has been assumed as equal to 4, the value derived by Grashof under certain assumptions made by Saint Venant. Estimates based on this would evidently be considerably in error. The value, 7-8, as here found is not surprising when compared with the value for steel, which is about 4.

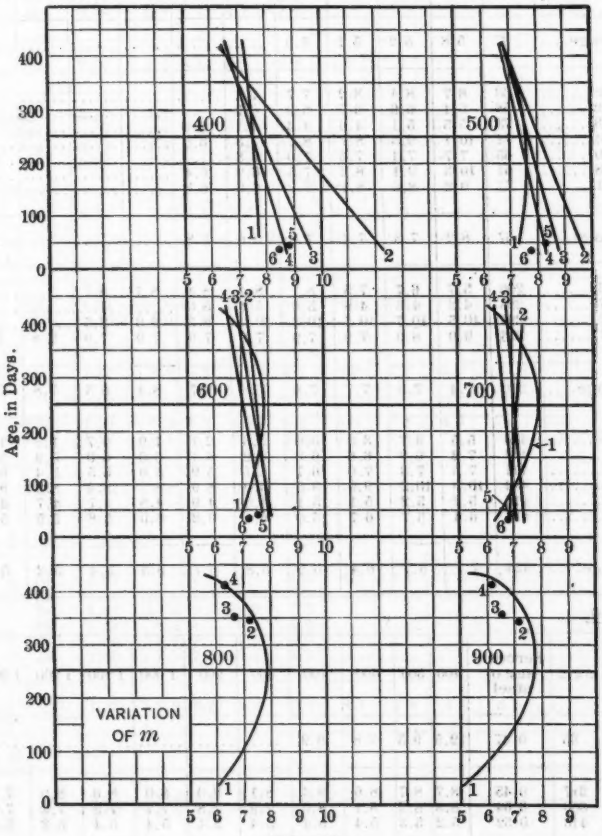

FIG. 18.

As compared with steel, concrete is lacking in density, has numerous small voids, and is granular in nature, with possibly a state of tension existing in the interstitial masses of cement due to the shrinkage of the latter in setting, under certain conditions. The first effect of applying a compressive stress to it would probably be to decrease

TABLE 7.—VALUES OF m.

GROUP 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.....	42	6.5	5.0	5.0	4.9	.....	.....	.....	.....	.....	.....	.....
114.....	35	7.6	7.5	7.0	6.6	.....	.....	.....	.....	.....	.....	.....
115.....	35	3.1	3.3	3.5	3.5	.....	.....	.....	.....	.....	.....	.....
Average....	37	5.8	5.2	5.1	5.0	.....	.....	.....	.....	.....	.....	.....
38.....	63	8.7	8.4	8.2	7.7	7.1	.....	.....	.....	.....	.....	.....
42.....	58	6.1	6.6	6.7	6.9	6.9	.....	.....	.....	.....	.....	.....
43.....	73	5.5	5.1	4.6	4.3	4.1	3.9	.....	.....	.....	.....	.....
44.....	74	10.1	9.3	8.7	8.1	7.3	6.6	.....	.....	.....	.....	.....
50.....	65	7.7	7.4	7.2	7.0	6.8	.....	.....	.....	.....	.....	.....
58.....	61	10.3	9.1	8.2	7.5	6.9	6.4	.....	.....	.....	.....	.....
61.....	77	9.2	8.6	8.1	7.9	6.9	6.3	.....	.....	.....	.....	.....
Average....	67	8.2	7.8	7.4	7.0	6.6	5.8	.....	.....	.....	.....	.....
46.....	296	5.7	6.7	7.5	8.1	8.4	8.4	8.7	8.7	.....	.....	.....
47.....	288	4.2	4.6	4.8	5.0	5.1	5.0	.....	.....	.....	.....	.....
53.....	295	10.5	10.7	10.5	10.3	10.0	9.5	9.0	8.5	.....	.....	.....
67.....	323	9.0	8.0	7.9	7.9	7.9	7.9	7.9	7.9	7.8	7.5	7.2
Average....	300	7.4	7.5	7.7	7.8	7.8	7.7	8.4	8.3	7.8	7.5	7.2
48.....	414	5.5	3.7	3.3	3.0	2.3	2.1	1.9	1.7	1.6	1.5	1.5
51.....	434	7.2	6.9	6.6	6.3	6.2	6.1	6.0	6.0	5.9	5.8	5.7
54.....	441	7.6	7.3	7.0	6.7	6.1	5.9	5.6	5.5	5.4	5.3	5.2
62.....	434	10.9	10.2	9.9	9.5	9.0	8.9	8.7	8.4	8.2	8.0	7.8
79.....	416	5.2	5.2	5.1	5.1	5.0	4.9	4.8	4.7	4.7	4.6	4.5
80.....	417	6.4	6.7	6.7	6.6	6.4	6.2	6.0	5.8	5.5	5.3	5.0
Average....	426	7.1	6.7	6.4	6.2	5.8	5.7	5.5	5.4	5.2	5.1	5.0

GROUP 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
111....	35	0.47	12.5	9.5	7.8	6.9	.....	.....	.....	.....	.....	.....	.....
71....	317	0.45	8.7	8.7	8.6	8.4	8.1	8.0	8.0	8.0	8.0	7.9	7.8
75....	300	0.54	8.3	8.2	8.1	8.0	7.9	7.8	7.7	7.5	7.3	7.0	6.8
82....	413	0.52	5.2	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.3	5.3	5.3
Average.	343	0.50	7.4	7.4	7.4	7.3	7.2	7.1	7.0	7.0	6.9	6.7	6.6

the volume by linear deformation in the direction of the stress, with little or no lateral enlargement. If this should actually take place, one would expect the value of m to be infinite at the beginning of application of load, and to decrease as the load increased, giving a

TABLE 7.—(*Continued.*)

GROUP 3.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
66....	74	0.72	14.4	13.3	12.6	12.0	11.6	11.2	.....	.....	.....	.....	.....
101....	29	0.71	8.7	7.8	7.1	6.2	.....	.....	.....	.....	.....	.....	.....
107....	35	0.78	5.7	5.1	4.7	4.2	.....	.....	.....	.....	.....	.....	.....
Average.	46	0.74	9.6	8.8	8.1	7.5	11.6	11.2	.....	.....	.....	.....	.....
64....	435	0.67	5.7	5.7	5.8	6.0	5.9	5.8	5.8	5.8	5.7	5.6	5.5
69....	323	0.72	7.9	8.0	7.8	7.2	6.6	6.3	6.1	6.0	5.8	5.7	5.5
72....	383	0.72	7.8	7.8	7.6	7.4	7.2	7.0	6.8	6.6	6.4	6.1	.....
81....	270	0.86	6.6	7.0	7.7	7.2	7.1	7.0	6.8	6.5	.....	.....	.....
Average.	352	0.74	7.0	7.1	7.2	7.0	6.6	6.5	6.4	6.2	6.0	5.9	5.5

GROUP 4.

59....	67	1.31	11.6	10.8	9.9	8.9	8.0	7.3	6.7	.....	.....	.....	.....
99....	31	1.16	6.0	5.9	5.7	5.3	.....	.....	.....	.....	.....	.....	.....
Average	49	1.23	8.8	8.3	7.8	7.1	8.0	7.3	6.7	.....	.....	.....	.....
63....	395	1.24	5.3	5.3	5.2	5.2	5.1	5.1	5.0	5.0	5.0	.....	.....
73....	428	1.26	8.1	8.1	8.0	7.9	7.7	7.4	7.1	7.0	7.0	6.9	6.9
Average.	411	1.25	6.7	6.7	6.6	6.5	6.4	6.2	6.0	6.0	6.0	6.9	6.9

GROUP 5.

56....	58	1.69	9.1	9.0	8.8	8.5	8.1	7.6	7.0	6.3	.....	.....	.....
92....	34	1.41	10.1	9.0	8.0	7.2	.....	.....	.....	.....	.....	.....	.....
93....	34	1.41	10.4	9.3	8.3	7.3	.....	.....	.....	.....	.....	.....	.....
94....	35	1.41	8.2	7.7	7.2	6.6	.....	.....	.....	.....	.....	.....	.....
105....	49	1.35	7.9	7.7	7.1	6.6	.....	.....	.....	.....	.....	.....	.....
106....	44	1.50	7.5	7.0	6.4	5.9	.....	.....	.....	.....	.....	.....	.....
Average.	42	1.46	8.9	8.3	7.6	7.0	8.1	7.6	7.0	6.3	.....	.....	.....

GROUP 6.

95....	35	2.04	10.4	9.6	9.0	8.4	.....	.....	.....	.....	.....	.....	.....
96....	35	2.02	9.9	8.5	8.0	8.0	.....	.....	.....	.....	.....	.....	.....
Average.	35	2.03	10.2	9.1	8.5	8.2	.....	.....	.....	.....	.....	.....	.....

curve such as is found in Columns 52 or 62. Generally, however, the curve runs from a lower value up to a maximum at a stress which roughly averages 275 lb. per sq. in., but which may be anywhere between 100 and 500 lb., or may not be clearly defined at all. The

points up to the 500-lb. stress are irregular in location, and it is often impracticable to lay a fair curve through them, indicating that the material is settling together somewhat irregularly and adjusting itself under the imposed load, not uniformly, as would a homogeneous mass, but somewhat spasmodically, until a stage is reached where the mass is thoroughly compacted, and all the initial stresses are relieved; from this point the deformation becomes quite consistent, as is shown by the values of m becoming quite regular, the points varying but slightly from a fair curve.

This same tendency to irregularity is shown slightly in the s_2 curve, but far more plainly in the e curve, where the points are irregularly placed, frequently showing a considerable break in continuity at some point. This irregularity of e seems to be without any law, except that it disappears at stresses of from 200 to 400 lb. There is no reason to suppose that slipping of the extensometer would be more likely to occur here than at higher stresses, and the fact that the values of e, which depend only on the s_2 readings, show an irregularity similar to that of m would indicate that this is not due to slip in the instrument.

It is the conclusion of the writers, therefore, that below about 400 lb. per sq. in., the concrete deforms under the load quite irregularly and without any law; that at about 400 lb. the causes of this irregularity are largely eliminated, and that thereafter the action conforms closely to a definite law which can be represented by a fair curve until some higher limit is reached; which limit lies well beyond ordinary working stresses, and is not defined by these tests. At this limit the destruction of the cement bond would begin, and the curves would become irregular again.

Conclusions.

From the foregoing results the writers deduce the following conclusions, as applicable to the materials used in these tests:

(1) The value of Poisson's ratio is approximately 8 for short-time and 7 for long-time tests.

(2) The value of e is in general below 10, only exceeding that value for short time and high stress.

(3) Spiral reinforcement does not, by its restraining effect on the core, perceptibly affect the values of m or e for stresses below 900 lb. per sq. in.

(4) Spiral reinforcement does not perceptibly assist the concrete in carrying the load within the limits of ordinary working stresses.

(5) The limit at which the first visible sign of failure occurs is raised by spiral reinforcement. This effect is greatest in short-time tests, and is perceptible for all percentages of reinforcement.

(6) The plain column fails abruptly, giving no warning of approaching collapse. The hooped column may be depended on to give warning of approaching failure at from 70 to 90% of the ultimate strength, when not less than 0.50% of steel is used.

(7) The deformation is quite irregular up to about 400 lb. per sq. in.; above this it is quite regular up to a limit which is not defined by these tests, but is well above allowable working stresses.

The tests indicate, but cannot be said to prove conclusively, that:

(8) The ultimate strength of the column is increased by spiral reinforcement by an amount roughly proportional to the quantity of steel used, up to the limit of 1.58%; and above this limit the ultimate strength is simply the amount which can be carried by the granular core supported by the spiral, but with no cohesion of its own.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By Messrs. Onward Bates, Jared How, W. H. Courtenay, V. K. Hendricks, E. W. James, W. W. K. Sparrow, William J. Wilgus, J. E. Gibson, H. C. Vensano, William W. Crehore, and C. P. Howard.

Onward Bates, Past-President, Am. Soc. C. E. (by letter).—The valuation of public utilities for the purpose of rate-making is one of the great economic questions of the present day in the United States, affecting as it does the value of investments aggregating thousands of millions of dollars, and the income from wages and dividends of some millions of people. The able report submitted by the eminent members of this Committee will carry great weight, and will doubtless be quoted back and forth by those who are engaged in the practical solution of the problems involved. This report, however, is admittedly incomplete, and is submitted with the reservation of the privilege to add to it. Consequently, it should not be taken as the last words of the authors. Mr. Bates.

The Government of the greatest nation in the world is committed to the policy of the regulation of big business, and incidentally of little business, and we have the example of men without business experience directing in detail how business shall be conducted, holding over the heads of citizens engaged in business the threat that the Government itself will take business out of the hands of its owners and conduct it as a Government monopoly. The notable present instance of this policy is the regulation of railway rates by the Interstate Commerce Commission, and it is in the minds of many people interested in rail-

* This is a discussion on the Report of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, presented to the Annual Meeting, January 21st, 1914.

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ways that a continuation of this policy will ruin this line of business endeavor, in so far as the interests of the men, women, and children who are railway stockholders are concerned.

The future of all public utilities is dependent on the rates which the public, through its Governmental instrumentalities, may establish from time to time. "Justice" will be the watchword of those who name the values and of those whose property is valued. The Committee, under the heading "General Principles Involved in Valuation for Rate-Making," writes:

"The primary requirement in the valuation of a public service property is that justice shall be done both to the owner of the property and to the public." * * * "The exercise of the power to regulate rates should be based on equity to both parties.

"It has been well settled by the highest Courts that the owner of such property is entitled to a fair return upon a fair value of the property utilized in or reasonably necessary to the service."

Justice, however, is often obscured. It appears from the foregoing quotation that the highest Courts depend for justice on the fairness of the valuation. It is to be expected, and it is certain to happen, that controversies will arise between the owners of property and those who place values on such properties, and it is only natural and altogether probable that partisanship and bitterness will be engendered between the representatives of property interests and of the authority which undertakes to establish the value of property.

At this point we meet what the writer considers one of the worst features of Governmental ownership—two parties disputing over values, of which, one party, with the prestige of Governmental authority behind him, is clothed with the power to do injustice, while the other party is practically helpless. Justice, as has been stated, depends on the fairness of the party to the controversy which holds the power of decision, and equity can only be secured by having the valuer fair and just and competent.

We are all prone to evil. Men are not perfect, and when engaged in a fight are inclined, during the heat of action, to forget the principles of equity and justice while endeavoring to secure an advantage over an opponent. There is an additional danger to vested interests to be expected from the appointment of incompetent valuers. This is the day of professional reformers and efficiency enthusiasts, most of whom are not under the conservative and sobering influence of personal investments which are at stake. At a time when the Government is planning an organization for the purpose of valuing public utilities, which will require a great number of men, most of whom should be experts, and with the condition that such qualified experts are generally in the employment of the public utility corporations, it is to be expected that men with bright minds and plausible language will find in this organi-

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zation a market for their talents. These talents may be very great, but, if not balanced by experience and a sense of responsibility toward the interests at stake, may be dangerous instruments in the hands of the Government, leading to great injustice toward its citizens.

An illustration of what may happen in this line is to be found in the seventy-eight questions submitted by the Interstate Commerce Commission to the railways as a preliminary to the granting of an increase in rates. These questions are too long to be quoted here, but the necessity for an increase in rates seems to have been lost sight of by the Commission, which has started an investigation entirely aside from the business question to be determined, with the only possible result that it will create a delay in the administration of justice.

If it be assumed that, in the future, railway rates will be, or may be, based on the valuation of the physical property of railway corporations, we will find a large proportion of the membership of this Society engaged in this work of valuation, and divided between a staff of engineers employed by the Interstate Commerce Commission for this purpose, on the one side, and the engineers employed by the railway corporations, on the other side. Engineers are already being lined up in these two classes, between which there will be great diversity of opinion on many points, notably on the questions of depreciation and land values. Under these circumstances it will be most unfortunate for the report of the Committee to be considered as anything more than an expression of the opinions of its distinguished authors. The writer is willing to be corrected if he is wrong in making the statement that the Committee was not appointed to make a report on valuation for the purpose of rate-making. Its function was "to formulate principles and methods for the valuation of railroad property and other public utilities." The Committee may find justification in making a report on "valuation for the purpose of rate-making," in its statement that "Valuations of the property of public service corporations may be made for a variety of purposes," and its report on this particular variety may then be considered as only one section of a general report, the other sections of which are to be furnished later.

The Committee goes on to say:

"It is, of course, true that the principles and methods of valuation for rate-making are in most respects the same as those involved in the valuation for other purposes."

Granting this truth, if the Committee had confined itself to a general report, it would have avoided the criticism of engineers, specially interested in rate-making, who differ with the Committee in some of its conclusions. The Committee's report is on "Valuation of Railroad Property and Other Public Utilities for the Purpose of Rate-Making." The immediate and most important application of the principle of valuation for the purpose of rate-making is found in the case of the

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railways; but the report seems to be based largely on experience with water-works and public utilities other than railways. The report would have been more acceptable if its treatment were limited to local utilities which constitute monopolies in their own limited territory, such as water-works, gas plants, electric properties, etc. As a matter of fact, the report is based on the condition of, and legal decisions relating to, such utilities; but it also covers in a desultory way questions of valuation of railway property, and really lumps the whole proposition to include them. It attempts to apply the same principles to circumstances and conditions which are entirely unlike.

The rates to be charged for gas, water, telephone service, local transportation, etc., in Chicago, are not governed or influenced by the rates permitted in St. Louis or Milwaukee. The same principles may or should govern; but there are no competitive influences or conditions. For the regulation of those local utilities which are non-competitive, or competitive only in a very limited way, principles of valuation may be formulated which will be approximately fair and just; but the situation with railways is entirely different. Whatever advantages may come from railway valuation, the proper fixing of rates is not one of them.

The Government valuation of railways is not declared to be for purposes of rate-making. Authority to make the valuation, after repeated requests by it, was granted to the Interstate Commerce Commission which wanted it for whatever value of any kind there might be in the information. The Commission has repeatedly declared that rates cannot be based on valuation, although it may be one of many elements to be considered under certain conditions. It would be just as fair to assume that the valuation is being made for the purpose of Government purchase and ownership, as to assume that it is for the impracticable purpose of determining fair rates.

The impossibility of basing railway rates on the plan of fair interest return, or even profit, on the value of the property, is so patent that it ought not to require any re-statement. The whole railway system is competitive, and the Government, just now, is bent on breaking it up into still smaller competitive units. Between certain terminal points there are half a dozen or more lines with different values, whatever principle of valuation is used in determining them. All must charge the same rate; and no road can, of itself, fix its own rates.

The existing rate structure is the product of commercial, competitive forces, and as a whole is found, even by regulating bodies, to be about right. It needs constant adjustment to meet changing conditions and to insure reasonable equality. Following a world-wide change in wages and all costs, the entire rate structure may need to be increased horizontally—as is the case at the present time—but no road or system, even if legally permitted, could raise a rate by itself,

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because its traffic would immediately desert it and go to other lines. This is not true of local public utilities, because there is practically no other available supply of service for the public.

If all the railways were in one system and ownership, rates might possibly be predicated on value of investment, and changing costs be met by changing rates, so as to insure an average net earning. The strong lines would then support the weak; but, with diversified ownership and the demands of all parts of the country for adequate transportation facilities, nothing of the kind can be done, and speculation regarding it is useless and misleading.

The Committee should either have differentiated clearly and omitted railways from its discussion, or should have omitted entirely that part of its report relating to "purpose of rate-making."

The questions involved in the valuation of public utilities for the purpose of rate-making are of such importance and of such direct interest to the members of the Society that the fullest discussion of them should be encouraged, and final conclusions should not be adopted until discussions are exhausted. With constant changes occurring, the time when discussions will be completed may never arrive, and final conclusions may be indefinitely postponed. Such a result ought not to be regretted. We must act according to our best present information, and continually move on to better conditions.

It should not be assumed that the questions to be solved in valuing public utilities for rate-making are wholly of an engineering nature. In fact, the engineer has only a limited function in their solution. The valuation of public utilities for the purpose of rate-making calls not only for engineering knowledge, but as well for the knowledge possessed by lawyers, by financiers, by real estate experts, by business men, and by accountants, all of whom should state the cases, from their points of view, before an attempt is made to formulate definite conclusions.

The work of valuing railway property, recently inaugurated by the Interstate Commerce Commission, will be done under certain disadvantages, some of which are:

A large organization, not trained to the work at hand, but gathered at one time from such material as is available, with the certainty that the railways will retain in their own service the experts who are fitted by experience for this work; the demand for results from this organization necessitating conclusions before this new occupation is capable of producing correct conclusions based on knowledge. In other words, results will be published before the business is learned, and conclusions will be to a greater or less extent untrustworthy.

There will be the interpolation of politics, directly and indirectly, into a fight, for power on one side and for existence on the other side.

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It is claimed by railway officials that the total value of railway property in the United States is $15 000 000 000, more or less. It is not true, as might be inferred from the assertions of politicians and of sensational newspapers, that this property is owned by a few magnates whose only concern is to rob the public. The railways, collectively, are a useful and valuable asset of the whole nation. They are owned by the holders of their stocks and bonds, mostly innocent investors, including men and women who have bought the securities with the accumulation of their savings, and children and helpless people are represented by the investment of trust funds in these securities.

The American Society of Civil Engineers cannot afford to place its endorsement on a report which may be used to increase or decrease the returns to the owners of these railway properties. If the report was absolutely conclusive, the Society could with propriety stamp it with approval, establishing the truth, without regard to its effect on any interest; but this is not the case. The principles claimed and the conclusions drawn will be disputed by a large number of the members of the Society. The Committee itself does not consider this report as final, for, in foot-notes at the beginning of the report and on page 74 it reserves the right to add to the report before its final publication.

This report should be received by the Society without any endorsement, good or bad. It is carefully prepared, and contains much valuable information. It will be interesting and useful when published in the *Transactions*, but it should not be assumed to be an expression of the Society as a body. It should simply be received with thanks. The reception and publication of such a report is an honor to its writers. The Society is careful to publish the statement in its *Transactions* that it "is not responsible for any statement made or opinion expressed in its publications." This statement, which applies to every paper published by the Society, should have equal application to the report under consideration, and should apply with special effect to it, for the reason that there will be great diversity of opinion regarding its contents. Even if it were considered necessary for the Society to take action regarding the approval of the report, it would not be right for such action to be taken at the Annual Meeting, which is attended by a comparatively small number of members. If a division is sought on the question of approving the report, it should be submitted to the whole voting membership of the Society for such action.

The writer has received a letter from Mr. Jared How, an attorney-at-law in San Francisco, criticizing the Committee's treatment of the subject of depreciation, and, being in full accord with Mr. How's criticisms, he offers (with the consent of Mr. How obtained by telegraph) this to the Society for its information, and as a contribution by Mr. How to the discussion.

JARED HOW, ESQ.* (by letter).—The writer's attention has been called to an advance copy of the report of the Special Committee of the American Society of Civil Engineers appointed to "formulate principles and methods for the valuation of railroad property and other public utilities" which is designed apparently to be presented to the Annual Meeting of the Society. The writer has been allowed only a casual inspection of the report, and was advised, of course, that what he learned from it was to be held in confidence. Nevertheless he feels at liberty to mention it and criticize it. Mr. How.

The writer paid no attention to the contents of the report excepting so far as it deals with the subject of depreciation. He had too little time to read the subject matter of that topic with care; but is impressed with the idea that the Committee takes an unfortunate attitude toward it.

The basal principle of the relation between a public utility, a steam railroad, for example, and the public, is that of contract. The right to construct and operate a railroad is not a general right which appertains to every citizen as such. It is a right obtained only through the grant of a franchise from the sovereign power. The grant and acceptance of the franchise create a contract, the consideration of which on the part of the public is executed by the grant, and on the part of the grantee is executory and rests in the obligation to exercise the franchise in the manner contemplated by the grant. So a railroad company, having accepted a grant of a right to construct and operate a railway as a common carrier, is under contractual obligation to operate it as such and to furnish adequate and safe facilities for the conduct of all business which shall come to it. In other words, it is under obligation to keep its road and equipment up to 100% condition for efficiency, and to operate it up to that standard so far as the needs of the public require service. The manifest difference between the relation with the public of a warehouseman or an innkeeper, for example, and the owner of a railway, is that while the use of the property of each is of such character that the public has an interest in it and therefore may regulate the exercise of the use for its own protection, the warehouseman and innkeeper are under no obligation to continue their properties in the public use, and the owner of the railway is under such obligation. Furthermore, the warehouseman and innkeeper are not under obligation to furnish ample accommodations to serve all who shall apply to them. Each of them is justified in refusing to accept offered patronage, on the ground that he lacks facilities to care for it. The owner of the railroad property is not thus justified. If his facilities are insufficient, he must provide himself with more. The only pos-

* Attorney-at-Law, San Francisco, Cal.

Mr. How. sible ground for the distinction is that the warehouseman and innkeeper have no contract relation with the public, and the owner of the railroad has such a relation; and his obligation growing out of that relation is to provide himself with such property and to operate it, when acquired, in such manner as shall be, not only inoppressive to the public, but, as well, adequate for the reasonable necessities of the public for the contemplated service for which his franchise was granted to him by the sovereign.

The fact that property of a railroad company is devoted to the public use does not make it public property. It is still private property, although it must be used for the service of the public; and, as such private property, it is under the protection of the provision of the Constitution of the United States that no State shall deprive any person of property without due process of law—and by "property" is meant, of course, the profitable use of property, as well as the title to it and dominion over it. Whether an owner of property is deprived of its profitable use seems to the writer to be clearly a question of fact to be determined by the application of the principles of economics, and not a question of law. The writer, therefore, comes to the consideration of the question of depreciation of railroad properties—the precise question being whether confinement of a railroad owner, whose property is maintained in 100% condition of efficiency, to a return on its depreciated value predicated on its cost newly produced, and not on its efficiency as an instrument, is, economically considered, a partial deprivation of the valuable use of his property.

The writer has before him, professionally, an estimate, made by an engineer of a State, of the present value of a line of railway. This railway was constructed as one task within a period of about 2 years, and cost about $5 000 000. The property was operated for some special traffic during construction. The valuation was made as of a date about 20 months after the road was turned over to the operating department. The State engineer finds its original cost and its reproduction cost; and then he estimates the "condition per cent." of each item of physical property, and averages them for the total, with the result that he estimates the present value—the value on which the owner is entitled to a return without interference by the State—as 95% of the amount of present cost of the property. If the property actually and reasonably cost $5 000 000, 2 years before this valuation was made, and could reasonably be reproduced at the time of the valuation for not less than that amount, as might well be assumed, then, if the owner shall be limited at the time of the valuation to a return on only $4 750 000, it seems to be clear enough that somewhere he has wholly lost, or has at least lost the valuable use of, $250 000. There is no force in the suggestion that he is entitled

to earn and lay aside this $250 000 during this period of 2 years in which the depreciation is alleged to have accrued. Even if he shall be able to earn it, he may not consider it as a return of a part of his investment and devote it to his private use as such. He is under exact obligation to use it in replacing the depreciating articles when they shall have depreciated to the point of ineffectiveness. In other words, because the proper maintenance of the property is the very first charge against the earnings of a railroad company, and because the property must be maintained ordinarily in perpetuity, money properly necessary for replacement of portions of the property when they shall have depreciated into inefficiency, and reserved by the company for that purpose, may just as well be said to be devoted to the public use as may the physical property of the company actually used in operation. It may, perhaps, be invested by the company until the necessity for its use arises, but if the investment proves unfortunate and the money is lost, it must be replaced, and, apparently, at the expense of dividends which otherwise might be properly payable; and, if the depreciation fund is invested and produces income, of course that income ought to be included in the general income of the company on the net of which the reasonableness of its rate of return is to be figured. Therefore, if the physical property is to be valued on a depreciated basis, obviously the depreciation fund necessarily segregated for the purpose of covering depreciation should also be valued, and a fair return should be allowed on both. If one builds a line of railroad 150 miles long, the cross-ties may all be laid in one year and may cost $325 000. Assuming the life of the ties to be 8 years, the railroad, in the eighth year of its existence, is none the less valuable to the public and none the less valuable to the owner, if he has on hand, and applicable to no other purpose, money sufficient to replace the cross-ties at the end of the year. Why then should he be limited to a return on the value of the property, less cross-ties, unless, indeed, he shall have added to that lessened value the amount of money which he shall have laid aside as necessary for furnishing new cross-ties? The writer cannot avoid the conclusion that if the owner is thus limited he is deprived of the valuable use of a portion of his property devoted to the public service.

Mr. How.

This is not much of an argument. The writer has not time to be as convincing as he is sure the soundness of his views on the question would permit. He writes because he thinks it will be calamitous if the Society goes on record in favor of economic practice, which seems to be utterly iconoclastic, and without a dissent.

W. H. COURTENAY, M. AM. SOC. C. E. (by letter).—Under the head of "Engineering", occurs the following statement:

Mr. Courtenay.

Mr. Courtenay.

"On railroads it is commonly estimated that the engineering cost will amount to 5% of the physical valuation of the property, exclusive of overhead charges."

The Committee proceeds to give certain specific examples of the engineering cost of other public service work, but does not give any specific examples of railroad work other than the subways in Boston and the Pennsylvania Railroad tunnels, both of which are out of the ordinary.

If the Committee has other data as to the ratio of the cost of engineering to that of construction on railroads, it would be desirable information.

Mr. Hendricks.

V. K. HENDRICKS, M. AM. SOC. C. E. (by letter).—This report covers the question of valuation in an excellent manner, getting down to the fundamental principles, and the writer considers the method proposed good for appraising property with a view of obtaining an equitable sale price. However, it requires a change in the system of accounting now in use on railroads, and as the same results can be obtained for rate-making purposes by using the valuation new, without changing the accounting system, it seems preferable to omit depreciation for rate-making purposes. It would be quite objectionable to change the system of accounting on all railroads.

In addition to this objection, it is believed that the use of the valuation new would cause less danger of errors through determination of the exact status of depreciation on each property, and would thus produce more uniform results, which would be more equitable to new and old properties alike.

By using the valuation new, it would be necessary, for the party calculating a rate determination, to consider each long-lived item separately, using the adopted percentage (which is practically uniform) corresponding to Columns 6 or 7 of the table showing the "Equal Annual Payment" method, and taking no account of the amount actually charged to operating expenses for the maintenance of such items. Short-lived items, which could not conveniently be given such special consideration, would be considered through the operating charges for maintaining such items, in the same manner as proposed by the Committee. By eliminating the actual depreciation for each individual property, the work of preparing the valuation, of all railroads, for instance, would be materially reduced and also the work of the "rate-maker", in that his calculations on long-lived items would be made from tables based on the expected life of the various items, and would be uniform for all similar properties, instead of requiring a different set of calculations for each year of any one property's life. This method accepts the theory contained in the Committee's report, and merely provides for a different method of application.

Mr. Hendricks.

The writer, therefore, recommends that, for rate-making purposes, the Society endorse the use of the valuation without any allowance for depreciation. This is an arbitrary ruling, but attention is called to the fact that in its report the Committee has made an arbitrary ruling in distinguishing between the method of handling long-lived items (tentatively assumed as having a life of 5 years or longer) and short-lived items, such as lining and surfacing of track, etc., the maintenance of which is just as much a restoration after depreciation as is the maintenance of the long-lived items.

Under the heading, "Application of Equal Annual Payment Method", it is stated that "the placing of additional ballast on a railway roadbed" should be charged to current expenses. The writer thinks this should read "the replacing of ballast on a railway roadbed", as new and additional ballast should certainly be charged against the capital account.

Mr. James.

E. W. JAMES, ASSOC. M. AM. SOC. C. E. (by letter).—In the large mass of expository and argumentative material produced during the last three years on the matter of valuation for rate-making purposes, only a small percentage can be dignified as contributions to the literature of the subject. The most prevalent fact—so common, indeed, as to be usual—is the failure of writers on the subject to establish any fundamental principles; and next, to confuse, sometimes in unexplainably simple ways, the elemental economic truths. The early days of many public service corporations display a tangle of promotion, corruption, lease, combination, purchase, and reorganization. Original records in many cases are lost. As a result, the discussion of physical valuation has been largely confined to consideration of the ultimate, technical details of some practicable method, to the neglect of important underlying principles. Valuation has been usually argued from the point of view of the material interests of the disputants. It has been only within the past two or three years that really serious conclusions have been arrived at. The report of the Committee, as presented, gives something lucid, fundamental, and unbiased, on which to hang discussion. The Committee deserves much credit for covering the subject in such detail, and at the same time in recognizing the basis in principle so essential to the whole matter. The report is a most valuable contribution.

At the same time, it was to be hoped that somewhat fuller exposition would profitably be made with regard to the economic and social principles that most certainly form the substructure of the whole valuation edifice. The Committee recognizes this idea when it states that:

"The primary requirement in the valuation of a public service property is that justice shall be done both to the owner of the property and to the public."

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This implies "fair rates" adjusted to produce "fair returns"; but it is obviously necessary to go back of this statement. It would not be just to close our eyes to past conditions, in any of their aspects, and rest our justice on present contingencies alone. Some papers have discussed at length the violation to property rights that would inevitably flow from any valuation that did not produce a fair return on the money investment of the present holders of stock. Such a valuation, of course, would do magnificent justice to the present stockholder; but, unfortunately, it would be saddling the public with a burden of fictitious values of all kinds for an indefinite future. This extreme case shows that the actual investment of the present stockholder is dangerously subject to sharp contraction in value; that such investments, heretofore seemingly sound, are to be a source of loss; that the investing public, like the proletariat buyer and more modest wage-earner, cannot look to the public for guaranty of all their past transactions. The investors took chances, naturally in the hope of adequate gains. It would be highly unreasonable, now, to remove all or nearly all the element of risk from such investments and assure to the stockholder for all the future a fair return on all the paper values he holds. The fact must be frankly recognized that someone is going to lose, and perhaps lose a great deal, in any adjustment of conditions flowing from rate-regulation on a valuation basis. It is inevitable that an older ground of justice than present conditions afford be sought and accepted as the really equitable and effective one.

In support of this, consider for a moment the recent events in connection with the passing of a dividend by the New York, New Haven and Hartford Railroad, and the insolvency of the St. Louis and San Francisco. As a result, perhaps, of the intricate financial maneuvering indulged in by the managers of these roads, the stockholders have had to accept some investments that are likely to turn out bad. They are going to lose some money, perhaps lose only paper values. Maybe they have been mulcted by trusted officials. In any case, however, whatever degree of truth there may be in any of these suppositions, certainly the general public should not be called on to relieve the stockholder of loss, or guarantee him for the future a fair return on funds out of which, frankly speaking, he has been cheated. So, it must be accepted that present justice cannot be done. Present holders cannot be entirely protected. It would be as just to have a public guaranty that all past real estate ventures by individuals in a given community shall result in gain, or at least in no loss. What the community should do in the future in this matter, however, is quite another affair.

This point of view probably makes the injustice loom large; but is the degree of probable injustice as great as it appears? Considering the holdings of stock in such roads as the Louisville and Nashville,

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the Pennsylvania, and the New York, New Haven and Hartford, it is probable that not more than a very small percentage of all public utility stocks that may be affected with loss as a result of rate-regulation on a fair plan, are held as *bona fide* investments by the people who, for instance, deposit in savings banks. A large mass of these holdings are held by direct, though in some cases now temporally remote, virtue of past manipulations. The holders took large risks and knew it. Their holdings do not represent vested interests of a sort that the public is bound for all time to hold sacred. In the past these holders have made huge gains; often their predecessors made huge gains. The profits they hoped for when they assumed their original risks have been forthcoming. Now, it appears likely that there will be a period of change, a time of reaction, perhaps some loss; but, in essence, the injustice will be of a blameless sort. In some cases innocent investors will lose, some homes will be impoverished, some lives saddened; but such painful facts are a part of all economic change, for good or bad, and are everywhere recognized in the common-sense fact that all investments cannot be successful. "Lambs" have been losing for years on the stock exchanges of New York, Chicago, San Francisco, and New Orleans. "Suckers" have invested millions of hard-earned, lonely dollars in wild-cat schemes of all kinds that appeared to them to be just as certain of returning substantial profits as the Pennsylvania or the New York, New Haven and Hartford Railroads. It is pitiable, but it is true.

A deeper-founded justice must be sought whereon to base an arrangement for fixing rates. It would, the writer ventures to suggest, have been profitable and illuminating had the Committee included in its report a more extended examination of the principles that are sure to be recognized sooner or later. Especially does this aspect of the general question appear inviting in connection with the most wise suggestion of the Committee, that:

"It would be entirely just and equitable, and it is highly desirable to provide by law that future public service properties should be valued on the basis of their actual reasonable existing investment."

The great value of this suggestion cannot be too soon recognized and acted on. No problems of any complexity would exist regarding such valuations, for the method of appraisal would be stated in advance. The experience that German municipalities have commonly had indicates that American concerns are quite ready and able to enter the field of furnishing public utilities when the investment is closely scrutinized by State or municipal authorities, when the rates are fixed, and when all net receipts above a certain permissible income on actual, reasonable investment are returnable to the State or city.*

* "Municipal Government in Continental Europe," Albert Shaw, pp. 52, 350, *et seq.*

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These conditions are considered satisfactory, and are met by American investors abroad; it is reasonably assumable that they would be acceptable to American investors at home. The sooner the conditions and limitations affecting future public utility valuations are prescribed, the better temper and security will be enjoyed by that part of the business world involved.

This recommendation of the Committee regarding the provision for making valuations of new utility properties, or for revising old valuations henceforth, indicates that, however much the Committee may believe in cost-of-reproduction-new as the most satisfactory method for making present-day valuations of old properties, it, nevertheless, is of opinion that actual reasonable cost to date of the property useful and usable in the public service would be the most just and satisfactory basis for valuation purposes. If the Committee recommends it for the future, it must at least accept its principle as sound; and therefore failure to use it for the valuation of old properties flows from considerations of expediency or compromise.

In spite of the fact that the Committee has chosen not to emphasize the fundamental principles, or to explain at greater length its opinions in this regard, it nevertheless arrives at generally sound conclusions relative to many of the matters which have long been in dispute.

There are, however, such fundamental principles, and although a general discussion of them belongs rather to the field of the economist, and may be considered by some as much too academic in nature to be of really practical value, still there is no denying that the closer we adhere to the course prescribed by sound and true principles, the more certain it is that substantial justice will be done to the greatest number. When justice is referred to as entering into the purposes or results of physical valuation for rate-making, an ethical principle is assumed and admitted, though perhaps unconsciously. This is an aspect of valuation that has not received the emphasis it deserves. A study of the economic and social phases of the subject may be made much more illuminating that it has been. It appears that such a study would provide a solvent for several of the problems now in dispute among writers of undoubted ability and experience. In the first place, the public attention has not been directed toward corporation control because it had nothing else to busy itself with. The movement toward physical valuation is not an erratic one. The public's point of view has of late years been changing. People are seeing that they have an interest in public service corporations different from that concerned merely with securing good gas, steady current, accommodating trolley service, or pure water. The new laws providing for physical valuation as a basis for determining rates for the use of public utility products or services are an evidence of this change of attitude.

That the public, the user, has been slow to recognize its real interest

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in public service corporations does not in the least lessen the rightfulness of that interest, or its substantial basis in fact. A savage has little or no ethical life; for, although an ethical principle may be absolute, *per se*, the strength of that principle applied depends on contributory circumstances of a nature to compel its recognition. Circumstances of corporation growth, the misuse of the power flowing from privileges granted to individuals by public official bodies, have at last forced the user of the public utilities to recognize his ethical right to consideration.

The interest of the user in the public service company is, of course, first of all, material. The company furnishes some service necessary to health or to the reasonable comforts of modern life, or something necessary to business. These services cannot generally be secured except from the public service corporation. For this reason the corporation is looked upon rightly enough as a quasi-public organization. It is no longer a private affair. In addition to this material interest, arising in the need of the community for fresh and pure water, light, heat, and transportation, there is the other, deeper consideration spoken of above. This flows from the material interest, and is best described as an ethical interest. It is not merely by equity or expediency that the user has rights in public service utilities. It is because he puts something into the business. What he puts in is, from the viewpoint of the user, fixed capital; he cannot withdraw it, and heretofore has had little or nothing to say regarding the management of it. Much of this fixed capital, supplied by the user of the public service utilities, is in the form of franchise privileges granted by the user, through his representatives in the local legislative body, for a period of years or in perpetuity. It is just as surely a vested interest as the money of the bond holder which has laid rails, strung wires, dug ditches, or erected pumping stations and gasometers. The fact that the consumer turns the management of his interest in the business over to the other party, together with the fact that when once these contributions are made they cannot practically be withdrawn, establishes the right of the user to fair treatment at the hands of the public utility company. Still stronger appears the user's right to fair treatment when it is considered that he is, in the very nature of the case, the residual investor. That is, under any plan to establish such rates as will provide a "fair return" to the money investor, the user must hold himself ready at any time to meet all demands for financial support involved in the operations of the utility. The user may not provide additional money capital in the early years of development, and may never indeed do so; but, if such becomes necessary, the user must, we admit, assume the burden of interest charges created by the required borrowing. Mutual obligations between the user and the producer follow naturally as a result of mutual interests.

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These obligations are as binding on one as on the other. Responsibility for fair treatment has in the past rested with the producer exclusively, because he has occupied the position of control. Unfortunately, the user has not been convinced, by the treatment that he has received, that his rights have had the recognition to which they are entitled.

A very interesting account of some of the increases in capitalization which have been made by railroads during the past 40 years, appeared recently.* Though the citations are doubtless extreme, their number is so large and the sums involved so enormous, that, had no other instances occurred, there is sufficient ground for the introduction of sudden and drastic control of the financial management of the railroads. At the same time, much praise is due to the operative management for their successful efforts to pay dividends on no less than $784 000 000 for which not a cent has been added to the value or earning capacity of the railroads. After extended consideration, Professor F. W. Taussig declares, "it is doubtful whether the whole mechanism of irregular and swollen capitalization was at any time necessary or wise."†

Nevertheless, it has been the user who has had to pay the bill, and, as a quasi-partner in the concern, it has obviously not been just to him to add paper capital which shall stand for all time and on which he must pay dividends.

As a result of these conditions, the present movement toward some degree of financial supervision has resulted. It is time that the relative obligations of both the users and the producers be understood. The railroads are by no means the only public service corporations involved, but they are by far the most important, both in relative value and in economic status. Further, they are to-day so generally the only corporations of a semi-public character whose life has been long enough to become seriously complex or whose records have been seriously impaired by neglect or loss, that a solution of the question of regulation as applied to railroads will, if satisfactory, serve as a solution of the more general cases.

The fact that the railroads hold so prominent a place among public service corporations, the added fact that among such corporations they stand almost alone in doing an interstate business, and consequently are the most prominent subjects for the exercise of control by the Interstate Commerce Commission, gives them a place in the discussion that cannot be deprecated. In accord with this condition, every development of practicable methods of valuation has held the railroad in chiefest consideration. It is doubtless due to conditions existing in the accounting offices of the railroads, that the general

* "The Plight of the Railroads," W. J. Lauck, *North American Review*, Jannary, 1914.
† "Principles of Economics," Vol. II, Chapter LXI.

opinion of those who have had to give consideration to these matters is strongly in favor of cost-of-reproduction-new as a basis of valuation for rate-making. Mr. James

As the writer has pointed out previously, the Committee recognizes the weight of opinion in favor of such a method, but at the same time supports the view that in the future the actual reasonable existing investment should alone be accepted as a basis of the same valuation. Apparently, the Committee has adopted cost-of-reproduction-new instead of original-cost-to-date merely as a matter of expediency, and in a spirit of compromise toward the confusion of conditions existing to-day among railroad records and old methods of accounting. In the writer's opinion, it would be much fairer and much nearer conditions of justice if the basis of valuation were made original-cost-to-date, just as nearly as it could be arrived at. In many cases this cost could be arrived at but remotely; nevertheless, it could be found in many cases as accurately as cost-of-reproduction-new. Nothing is gained in compromising on a point of principle. Rather, first, establish the principle. If it is advocated as good for all the future, it must be good enough for all the past. If the principle is established as applicable to the past, the law can then legalize it for the future. Indeed, the difficulties of uniformly applying a set of principles to conditions that the past hands down to our time are an argument for statutory action. In the presence of these difficulties it will be necessary to adjust methods of application. This can be done. It is much better to adjust the application in particular cases than to sacrifice a principle. If attention is given to that principle suggested above, that the user has, by virtue of existing conditions, an ethical right in the property of any public service corporation that acts under a franchise or to which government has transferred at any time the right of eminent domain, it will be seen that there is only one just basis of valuation. That is cost-to-date, actual, reasonable investment. The investment does not necessarily include all the money spent in creating the utility. It is no part of the user's duty to secure to the investor a return on funds spent unwisely, unnecessarily, or in any improper way.

Of course, those who support the position taken by the Committee cite the fact that complete records for cost-to-date do not exist in numerous cases. Such contention is doubtless true; but, admit this, adopt the cost-new method, and at once uncertain conditions are met, and details are prescribed that are really an effort to approximate cost-to-date. Assuming that the records are lost or imperfect, what is the essential difference between an appraisal made in an attempt to approximate original cost-to-date and one based on cost-of-reproduction-new in which "the original conditions should control," and in which costs "should be based upon physical conditions existing

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at the time the various portions of the property were built?"* The Committee recognizes again that the original cost is the one to be approximated, when, in its summary† it suggests that "average prices prevailing during a period of from five to ten years prior to the date of valuation should be used." In this way the present and the past are made to overlap, and just when this shall be done and to what extent is itself a nice question. For who shall say when the old becomes old, or how much of the new remains new? After all, is not the question but little more than one of definition? Does not the Committee resort to the method it considers less desirable in order to correct too faulty results produced by the method it considers best? Is not the valuation suggested by the Committee, under any name one pleases, after all, the actual reasonable investment represented by the properties under consideration, as nearly as it can be found? It appears to be, so it should be fairly met. In the ethical principle suggested above there is sufficient cause for adopting such a basis of valuation.

Furthermore, this ethical principle is a solvent of several problems that excellent authorities now disagree about. This disagreement, as so frequently is the case, arises in all probability because there is no common ground of principle on which to rest the discussion. If we recognize the ethical right of the user in the public service corporations, we have at once the principle we need. Consider for instance the matter of land valuation. The Committee, after writing off depreciation, sees that the appreciation presents a perplexing question, and states:

"The Courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value."

Would the Committee hold that the public should receive any allowance due to a settled, stable grade, as contrasted with a new, green embankment? Consider this matter of land rather in the light of the ethical principle involved. The difference of opinion is really over the unearned increment. The usual and principal argument to support the inclusion of the unearned increment is that the public service corporation should not be denied what every land holder is allowed. Professor Taussig adopts this same position,‡ but points the way to an answer. Of course, if a valuation were being made for sale, taxation, or rental purposes, the inclusion of the increment would be right. If a utilities corporation chooses to rent or sell, it doubtless

*"Committee's Report," pp. 18, 15.

†"Committee's Report," Section 9, p. 71.

‡ *Supra.* Vol. II, Chapter LXI.

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is entitled to take advantage of the unearned increment, if it can find a purchaser on such terms; but the unearned increment of the land does not represent an investment of the producer but of the user. The increment arises from the development of the land contributory to the railroad. It is greatest in sections where the development is greatest. It is in essence a contribution of the community. The community is nothing more nor less than the user in an aggregated form, and as we have seen, the user is a co-investor with the producer in public utility properties. Obviously, no part of the return on the user's investment should be included in a "fair return" to the producer. It follows, of course, that lands donated, whether by a person, by a commercial body, or by government, should not appear in the valuation at all.*

The Committee itself may have had some misgiving with regard to the decision to include unearned increment. For the existence of a doubt is suggested in the expression used: "that if it be admitted that the [acquired] property is to share in the increase, etc."

The recognition of an underlying principle would not have led the Committee to decide that donations of either money or land by a community should be capitalized and made a basis of dividends to be paid by the community.

It is, of course, unnecessary to state that all legitimate expenses incurred in connection with the acquisition of land, all damage, transfer, and severance costs, should be included as a part of the cost of the land to the corporation.

Likewise, all necessary engineering charges, legal expenses, interest during construction, just exactly as during any later time, should all be allowed, and the recognition of an underlying ethical principle supports this view. Why, then, does the Committee say that "the amount of the overhead charges applicable to any property should be determined on the basis of the prices prevailing at or near the time of the valuation, but under conditions existing when the plant was created?" It will be as difficult to approximate the conditions existing twenty years ago as to approximate prices paid twenty years ago, and probably much more so. The aim should be to approximate both conditions and prices at the time of acquisition, and, indeed, the Committee probably sees this when it says, "the proper purpose of a cost of reproduction estimate of the value of real estate is to ascertain the cost of acquiring such real estate under the conditions existing at the time of its acquisition."

The Committee goes on to recite some special values attaching to land. Undoubtedly, if such values were recognized at the time of acquisition, and if a corporation was forced to pay a price for them,

* Similarly, bonds guaranteed by counties, a procedure not at all uncommon in the Southern States in railroad promotion, have no place in a valuation for rate-making purposes.

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such values should continue to be recognized in stating or approximating the original cost; but any law regulating valuations of future properties should rigorously exclude such values. For, obviously, such special values do not attach to the land; they come into existence in the land only when the utility enters upon it. The Committee's stand in this matter is undoubtedly right.

When the question of excess or deficiency of past earnings is considered, our attention is again directed to the advantage of emphasizing some principles that should control in establishing methods of valuation for rate-making. The Committee suggests that deficits occurring in the development period, if covered by the producers, should be capitalized, and this suggestion is well founded; but, if such deficits are made a basis of future dividends to the corporation, why, on the other hand, should not past excess earnings go to offset such development deficits? Surely, if the earlier records, covering transactions and financial conditions during the first years of existence, are to be considered adequate as a basis for determining development costs, then subsequent records, for years when organization has improved and crystallized, may properly and reasonably be considered adequate to determine the excess earnings.

Two very interesting matters are presented for consideration by the Committee relative to efficiency of operation and special advantages in site or conditions of production. No definite suggestions are advanced, however, with the view of introducing these elements into a physical valuation, beyond a general recommendation that they receive due consideration at the hands of the Courts and of the regulatory commissions. The second of these questions is one of economic rent, and has all the difficulties attaching to such. It involves the really serious question as to what poorer, less efficient organizations are to do, if rates are adjusted for highly efficient competitors in the same territory. In the nature of things, the poorer road would lose business unless it meets the competition. On the other hand, there is a serious situation created if rates are adjusted for the efficient road so that when applied to the inefficient road adequate revenues will be produced to operate and pay returns. There is a solution, but no one known to the writer has had the temerity to suggest it in this connection. The solution is to adjust rates to make the less efficient or less favorably located road pay the minimum return. Adopt a sliding scale of fair returns and permit the stronger road to charge the same rates as the weaker and pay to the State, in form of taxes, all but the necessary revenues to operate and pay a fair return according to the sliding scale. The higher permissible rate would then save the weaker road, and be sought by the stronger, and the sliding scale would preserve the required incentive to increased efficiency in the case of both, and prevent excess earnings.

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Obviously, such matters are due for serious consideration at a much later time, when regulatory methods are more advanced than at present; but such problems are certain to demand solution sooner or later. Those analysts of the present situation who are most disinterested, who see the tendencies of our time writ large and growing imperatively to demand a stronger control, do not hesitate to declare that the success of democratic government depends on greater discipline, more restraint, and fairer adjudication in all the relations of large-scale production with the general public.* Such control is not impossible. It exists in some nations of Europe. Such discipline as may be necessary does not necessarily destroy the incentive to save and invest, for a much severer discipline in Germany and Austria than will be developed here in many years does not do so.

The great question left is that of a fairer adjudication. This demands action in our time, change from present conditions; and all change hurts. The railroads constitute undoubtedly the greatest single development in the economic life of the western hemisphere. They are largely responsible for the general industrial development of the United States, and have played no small part in producing the seriously perplexing questions of regulation of public service companies. It would be a national disaster to hamper the railroads by a maze of restrictions regarding their operation, and great care must be exercised in this respect. Some States have already, doubtless, gone too far. Railroad operating managers are more expert in such matters than legislators; but control and supervision of railroad finances must be accepted as imperative. Such control is eventually to be continuous, and, in moving toward it, the chief consideration should be given to the principles underlying the elemental relations of the public and the corporations. Past wrongs must, so far as possible, be righted; but that no complete adjustment is possible is generally recognized. The greatest good can no doubt be effected by sane constructive legislation providing for control. On the success of this movement rests no inconsiderable part of our future national prosperity and progress. Engineers, as managers and as men of affairs, are wonderfully well equipped to handle these problems, and can hardly give them too much attention.

Mr. Sparrow.

W. W. K. SPARROW, ASSOC. M. AM. SOC. C. E. (by letter).—This report is most complete and exhaustive, and is certainly a very valuable addition to the literature on this subject. The Committee deserves the thanks, not only of the Society, but of all persons interested in the valuation of public utilities, for the time and thought it must have given to the preparation of such a report, covering as it does prac-

* "Principles of Economics," F. W. Taussig, Vol. II, Chapters LX to LXIII.

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tically all the basic and fundamental principles underlying the valuation of public service corporations.

The writer has not had the time to digest the whole report thoroughly, and will confine himself at present to the question of development expenses and land values.

The Committee defines the term "Development Expenses" as "the investment necessary to put the plant into successful operation and to create revenues that justify its construction," and prefers to use this term to that of "Going Value." The writer fully agrees with the Committee that the term "Going Value" has been given so many interpretations by the Courts and others that it is pretty generally misunderstood, and its use is inadvisable.

The Committee is of the opinion that, except in the case of a losing venture, the development expenses, as already defined, should be included in the cost of the property. The writer has given some thought and study to this question, and is not in agreement with this recommendation. It is understood at the outset that the report of the Committee, and therefore any discussion on it, relates to valuation for the purpose of rate-making only.

Development expenses may be divided into two classes:

(1) Expenditures made to create or attach business;
(2) Loss of dividends, and loss due to the earnings being insufficient to pay operating expenses.

The principle involved in the Committee's recommendation is that such expenditures as are classified in the first division and such losses as are classified in the second division constitute a property investment in the business, and that as such the general public is required to pay a return on this investment, or, in other words, that the company is entitled to capitalize such expenditures and losses on the same ground that it is entitled to capitalize interest paid on capital during construction.

Does any live well-managed property ever reach the point in its life when it is not necessary for it to expend large sums in developing and acquiring new business? On the contrary, is it not true that the larger and more prosperous a business becomes, the larger these expenditures become? Has any one ever heard of a company charging such expenditures to capital account and asking a long-suffering public to pay a return on such investment? Such expenditures are universally recognized and treated as a part of the operating expenses, and the writer can conceive of no good or logical reason for treating such expenditures, made at the beginning of the life of a business, in any different manner than that in which they would be treated if made during any other period of the life of the business. They are operating expenses

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first, last, and all the time. Recognizing this fundamental principle, the next point to consider is whether a company is entitled, as a legal right or in reasonable equity, to capitalize deficits in its operating expenses incurred during the first few years of its life. If such deficits are capitalized and treated as part of the investment of the company on which the public must pay a reasonable return, it logically follows that the return for depreciation, which is based on the total investment, will be increased in proportion to the amount of these losses, and, as a result, the public will in time, not only amortize such losses, but continue to pay a return on them.

The writer is not aware that any Court has laid down the principle, which this recommendation of the Committee involves, that the public should guarantee a company against loss from operation, and he doubts if such a principle will ever be adopted, but he feels sure that no Court will ever lay down the principle that such losses constitute a legal investment in the business of the company on which the public is obligated to pay a return. Although refusing to admit the principle that the public is obligated to refund the losses of a company made in the first years of its life, the writer believes that where a company, in the prosecution of legitimate business, has contracted losses in the starting of its business, or where the returns have been insufficient to pay a fair return on the investment, and the company can conclusively prove that it has not recouped such losses in later years, it would be fair and just in fixing the rates to make the return such that the company would be recompensed for such losses. In other words, the writer cannot conceive of such losses as an investment in a plant or business, but thinks that it is a condition which may justify a higher rate than would have been the case had no such losses been incurred, until such losses have been amortized.

The writer considers that if the principle were adopted that the deficiency, representing the difference between the amount actually earned and the amount which the corporation is entitled to receive, under the basic principle that it is entitled to a fair return on the fair value of its property, is to be capitalized and treated as an investment on which the public shall pay a fair return, then, as a corollary, the surplus, representing the difference between the amount actually earned and the amount which the corporation is entitled to receive under the same basic principle already outlined, should not only be returned to the public, but the corporation should continue to pay the public a fair return on this amount. The writer believes that on past records the public could well afford to adopt this principle.

Land Values.—In view of the physical valuation of the railways in the United States, which is about to be made by the Interstate Commerce Commission, this question looms up very large, and to

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the writer's mind overshadows all other questions in the valuation of public utility corporations. The tremendous importance of the question can be better appreciated by a reference to one or two recent cases involving this question. In the Minnesota Rate Case the total original cost to the Northern Pacific Railway Company of the terminal properties in Minnesota was $4 527 229. The Master in the United States Circuit Court allowed a value of $17 315 869. The total original cost of the entire system was found to be a little more than $312 000 000, but the cost allowed by the Master on the reproduction basis was more than $452 000 000, the difference, $140 000 000, being principally the increased value of the land.

In the Western Advance Rate Case, the Burlington claimed a return on a present value of $530 000 000. Commissioner Lane found that the amount of the original investment was only $258 000 000, and that about $150 000 000 of the Burlington's claim represented increase in the land values.

In the appraisal of the Northern Pacific Railway by the Washington Railroad Commission, H. P. Gillette, M. Am. Soc. C. E., Consulting Engineer to the Commission, placed the original cost plus improvements at $75 457 893, and the cost of reproduction new at $103 613 442. The difference is mainly due to the increase in the value of land.

These cases are only typical of many others, and they illustrate powerfully the magnitude of the question under discussion.

The claim on which the right to capitalize the increased value of the land is based, is that the practice in recent years has been in favor of the reproduction cost less depreciation, and that the fair value on which the return is to be made is the present value rather than the original cost, and that, on this basis, land must be treated in the same manner as the other property of the corporation.

If we study the history of valuation methods and Court decisions, it is easy to see how the reproduction cost or present-value theory came to birth. In the first cases which came before the Courts, the utility corporations claimed a return on the amount of stocks and bonds outstanding, which, it was argued, represented the original cost plus improvements, or the investment in the property. The Courts appear to have seen very clearly that to allow a return on such a basis would be unfair, for the very reason that such stocks and bonds in many cases included water, and did not represent the investment in the property. At this time there did not appear to be any difference of opinion in the minds of either the corporations or the Courts that the return should be based on the investment, and the only reason for the Courts' refusing to allow a return on the outstanding stocks and bonds was that they did not represent the *bona fide* investment. In searching for a way out of the difficulty, the reproduction-cost and present-value methods were devised. At the time it does not appear to have been

foreseen by the Courts that in avoiding the frying-pan they may have fallen into the fire, and that much greater injustice may be done the public in the application of these theories to the unearned increment of land. Mr. Sparrow.

Land differs very materially from the other property of a utility corporation for the reason that it seldom if ever depreciates. There may be times when the value of land does not increase, but, if taken over any considerable length of time, it will in nearly all cases increase. In the case of a utility corporation, it will invariably increase, and that enormously. Here, therefore, the application of the reproduction theory is very one-sided, and is as correspondingly unjust to the public as it is advantageous to the company.

It is recognized and accepted beyond further argument that the consumer must pay such a return to the company as will cover the amount of the depreciation of the company's property. This being so, and the consumer being debited with the amount of depreciation, he must, as a matter of substantial justice, be credited with the amount of the appreciation.

This contention is supported by the general rule laid down by Judge Hough of the United States Circuit Court in the case of the Consolidated Gas Co. *v.* City of New York. Judge Hough says in part:

"Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the case at bar."

Commissioner Maltbie has attempted to consider appreciation in value as income, canceling it against depreciation in other property, but the writer believes that the same result would be arrived at more logically and with greater simplicity by appraising the land at its original cost. The railroad is a public servant. It has been thus defined by the Supreme Court, but the writer prefers to consider it as the agent of the public.

In the case of Smyth *v.* Ames, Mr. Justice Harlan says:

"A railroad is a public highway, and none the less so because constructed and maintained through the agency of a corporation deriving its existence and powers from the state. Such a corporation was created for public purposes. It performs a function of the state. Its authority to exercise the right of eminent domain and to charge tolls was given primarily for the benefit of the public."

To the writer, the answer to the whole question is contained in this weighty and forceful declaration. If a railroad company or other public utility is the agent of the State, performing a function of the State, and in that guise is vested with a governmental authority to acquire land for the purpose of serving the public, can this fundamental relationship, to which the company owes its very existence, now

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be thrown aside, and the agent demand from the principal the increased value on the land it was allowed to acquire for the purpose of serving and benefiting the principal.

The land was never granted to the railroad company for real estate speculation. It was granted for one purpose only—railroad purposes, and for the benefit of the public.

If the rate is insufficient for the service performed, and does not yield a fair return on the money devoted to that service, then it is an argument for an increase in that rate, and, in justice to the company and in the interest of the public, the rate should be increased; but, if the rate is sufficient for the service performed and does yield a fair return on the money invested, then can it be considered as an argument for an increase of that rate that the real estate has increased in value and a further toll be imposed on the public to whose service this real estate was dedicated?

The writer is strongly opposed to Government ownership of public utilities, but he is a firm believer in Government ownership of land, and that the Government, State or Federal, as the case requires, should acquire all land necessary to the construction and operation of a public utility and lease such land at a fair return on the cost of acquiring it to the company, such rental being charged to the operating expenses of the company and paid by the consumer. If such a principle had been adopted from the start, the question of the unearned increment could never have arisen, as the increase in value of the land would have been credited to the account to which it rightfully belongs, that of the public.

Mr. Wilgus.

WILLIAM J. WILGUS, M. AM. SOC. C. E. (by letter).—Although the writer has the gratification of knowing that the views expressed in his recent* paper on the physical valuation of railroads are substantially in accord with those contained in this very complete and admirable report, he finds himself unable to agree to the proposition that, for rate-making purposes, depreciation should be deducted from the cost of reproduction new.

The Committee considers that depreciation should be divided into two classes, the first consisting of low-cost and shorter-lived items of property to be repaired or replaced through charges to current expenses; and the second consisting of high-cost and longer-lived items of property, to be renewed through charges to capital.

Therefore, apparently, it is the intention of the Committee that the first class is to be treated as an operating expense and the second class as a restoration of capital to the investor.

If this is a correct understanding of the Committee's position, the sum that it proposes shall be taken as the basis for rate-regu-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

lation is not "cost of reproduction new less depreciation," but rather "cost of reproduction new less the portion of depreciation that applies to high-cost and long-lived items." In this event, the portion of depreciation that is to be taken care of through current expenses is not to be deducted from cost of reproduction new. Mr. Wilgus.

Even though common practice and the rules of regulatory commissions should permit the charging of replacements of a large nature to capital account, which they now expressly prohibit, it is difficult to see how in practical every-day railroad accounting and maintenance it will be possible to draw a well-defined line between the two classes of depreciation.

The Committee says:

"As a general statement, it may be said that the basis of expectation of life should be used for all important items of property having a life of more than, say, five years, except where it may involve unwarranted expense for accounting or be impracticable to separate what would properly be, under the Equal-Annual-Payment Method, charges to current expenses and to capital."

The writer believes that on railroads with their vast multitude of parts of varying and uncertain lives, the exceptions mentioned would prove the rule and the plan would fall. Rails and ties on sharp curves or of inferior material or subjected to heavy traffic, may have a very short life, though at other places less exacting conditions will make for a greatly lengthened life. An increase in the weight of motive power suddenly may cause bridges to become obsolete, whereas a change from steam to electric locomotives, with their lessened weights, may as sharply reverse the situation. Buildings and wharves under certain circumstances or policies may be continued in use indefinitely, through repairs and remodeling; and under other conditions may be replaced with new structures; or they may be abandoned. The instances may be multiplied *ad infinitum* to show that the drawing of a line between the two classes of depreciation would prove to be impracticable.

With the admission by the Committee that, because of certain difficulties, only a portion of depreciation may with propriety be deducted from cost of reproduction new, the grounds for asserting as a principle or fact that a depreciation allowance is a return of a part of the investment would seem to lose force.

The crux of the question appears to lie in the requirement of the Interstate Commerce Commission and the various State regulatory bodies that upkeep shall be charged to expenses and not to capital. As long as this situation continues, depreciation plainly is a liability of the owner to be replaced by him out of the allowance made for that purpose in the rate, or in case of misuse of such

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allowance, to be made good by him out of the part of the rate assigned to him for profit.

In this connection, sight should not be lost of the impossibility of maintaining a railroad in a new condition. A well-maintained property in time reaches an average condition, say, 85% or 90% of new, which remains practically stationary. In such cases an appraisal with depreciation deducted will show a loss of value, though, as a matter of fact, the real worth will be greater. For instance, no one will deny that the Pennsylvania Railroad between Jersey City and Philadelphia is more valuable for the purpose for which it is used, than a brand-new unseasoned parallel line having the same physical characteristics. Depreciation in cases like this is merely a measure of the cost of securing a seasoned railroad and may be considered as belonging in the same category as other legitimate overhead expenses.

The importance of this depreciation question is so great that it is to be hoped that the Society will not lightly lend its sanction to any view that will not successfully bear a critical analysis.

The Committee considers that adaptation and solidification should have no place in the valuation, unless they have involved the expenditure of money. As no such rule is proposed in the cases of donations of right-of-way and other property, it hardly seems logical to exclude the enhanced value that comes with time to the seasoned railroad, nor does this course seem equitable when the stand is taken that depreciation due to the action of the elements and natural causes shall be deducted.

Mr. Gibson.

J. E. Gibson, M. Am. Soc. C. E. (by letter).—The Society is fortunate in obtaining this valuable report, and the gentlemen composing this Committee are to be congratulated on their thorough and painstaking work. It is to be hoped, however, that the Society will give careful consideration to this report and before presenting it to the public as its report, will consider every phase of the matter, so that all parties in the proposed valuation, if conducted under the Society's rules, will have justice and equity. Undoubtedly, any rules that the Society may formulate for procedure in valuation cases will carry great weight with the public and the Courts. The report, the writer believes, does not insure complete justice and equity to the privately owned utilities.

The report is very comprehensive, but there are several points on which the writer cannot agree with its conclusions. It has running through it that deference and awe that all have for the majesty of the Court. The writer does not in all cases share this deference, as he recognizes the Court at best as only human, and as likely to err as others. Necessarily, the Court lives in the past, and its actions are controlled by precedent, whereas the engineer must of necessity

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be a pioneer and establish his own precedents and landmarks. This being the case, the Courts have erred unintentionally and honestly in many of their decisions in the valuation of public utilities. To correct these conditions and to place before Courts and commissions proper rules for procedure is one of the functions of the members of this Society. Men trained in engineering subjects and having wide engineering experience in construction, in conjunction with men trained in financing and operating public utilities are certainly more competent to pass on the valuation of these utilities than those trained in the law, such as the Courts.

Purposes of Valuation.—The Committee states in one of the opening paragraphs of the report as follows:

"It is sometimes held that valuations for all of these purposes should give identical results. This view cannot prevail if the laws of the different States and certain decisions of the Courts are complied with."

It seems to be a common belief that the value of a property depends on the purposes for which the valuation is made, but the longer this is considered the more one is forced to the conclusion that the true value of a property is independent of the purposes for which the valuation is made. If this premise be correct and the decisions of the Courts and the laws of the various States are antagonistic to these premises, then it is our duty as a body to take all reasonable and just measures to modify and correct the laws and decisions of the Courts. The value of a property for the purposes of sale should be the same as that for the purpose of rate-making or taxation; for, should the purpose be that of sale, and a sale be consummated on the valuation made, then the new purchaser should be entitled to receive a proper return on the purchase or valuation price, and this return, in justice to all parties, should be neither greater nor less than that on the valuation of the property before sale for rate-making purposes.

It is realized that a property may have a sentimental value, but no purchaser is willing to pay a greater price because of such sentimental value. On the other hand, under certain conditions, a purchaser will pay more than the apparent real value of a property owing to a latent or potential value, which, by his business ability or connections, he can transform to an active value, but, in so doing, he converts the latent value to the uses of the public, and is therefore entitled to a proper return on this increased value.

The Committee touches on what would be termed the negative phase of this latent value where it speaks of a property, which, under certain abnormal conditions, may never be able to earn a fair return on the investment, and says "such a property may be classed as a losing venture". In valuing a property, should it have a latent value

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due to strategic position or other natural facilities, this value should be added to the physical value, the same as an appreciation in the value of its real estate, and, on the other hand, by unfortunate location, wasteful construction, or otherwise, should the property be unduly expensive, then this inherent and excessive cost should be deducted from the valuation, the same as depreciation on the physical plant would be treated.

Methods of Determining Physical Value.—A board of public utility commissioners or engineers should strive to place themselves in the position of the original builders at the time the plant was conceived. It has been the writer's observation that with but few exceptions all engineers and contractors are optimists, and are always willing to undertake to do for less money, and in a shorter time, more work than they can actually perform. Therefore, in valuing an existing plant, and notwithstanding that they have a life-size plan of the work to be valued, the appraisers are more than likely to lose sight altogether of the contingent expenses and items that go to make up a completed whole—such as the difficulties of labor, non-delivery of machinery, conditions of weather, difficulties of financing, and the other multitudinous delays that occur in the construction of extensive works. They should always be very liberal in their estimates and criticisms of the property, and give the benefit of every doubt to the owner of the property. To this end they should use great care and diligence in reaching their final conclusions, and, for this purpose, should always seek the advice and counsel of men who have had experience and training in the actual construction of such works. It is obviously unfair to ask a railroad engineer to pass on the cost of a water-works property. He (the railroad engineer) would unintentionally, but honestly, undervalue an embankment for reservoir purposes, as the methods of construction are entirely different, and, in the same manner, it would be unfair to the railroad property to ask a water-works or hydraulic engineer to value a railroad property. It seems, however, to be the practice of most of the present public utility commissioners to employ one chief engineer and expect him to pass on all classes of utilities. This procedure, in a great many cases, works an injustice on the majority of the utilities in the State. Again, on account of the niggardly policy of some of our legislatures in the payment of its officers, the commissioners are very often compelled to accept the services of men who have not had the necessary experience in the actual construction and operation of public utilities, and therefore do not realize the cost of performing satisfactory service.

Unused Property.—This is a rather difficult subject, but if the property that is not in use has been purchased with a view of future extensions and use, and this future use or extension will be required

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within a reasonable time, the company is entitled to a return on this property; and, in line with this reasoning, a public utility is entitled to a return on a plant which is built larger than the immediate needs require. Of course, the appraisers should use their discretion as to whether this unused property or over-built plant is beyond the reasonable requirements and needs of the future, but, even in this case, the utility should be given the benefit of the reasonable doubt. A water-works utility company, for instance, is certainly entitled to construct a plant of sufficient size to meet the public need for a period of at least 10 years in advance of the immediate requirements, and any outlays of money looking toward the extension of its piping system, pumping facilities, or water supply, with this object in view, are certainly within legitimate business requirements, and one which the public should demand of the utility.

Present or Original Conditions.—The board of appraisers, in considering the present conditions of the plant, should also consider the conditions existing previous to the commencement of construction; otherwise, the utility might be done a great injustice.

The writer does not agree with the Committee regarding the increased value of the plant due to public improvements, etc. Under existing laws, the holder of real estate or other property is entitled to the unearned increment of value from his property, and the same should hold for a public utility. Because the utility has been given something outright does not relieve the public from paying a fair return on this property, provided it receives the benefit of the use of the property: Because A buys an auto truck for $10 000, and then turns it over to B for nothing does not impose on B the operation of this truck for the benefit of the public without a return. This is a parallel case to that of a public utility which has had service connections and rights of way donated to it, or that of the public constructing highway improvements in its territory. The writer, however, believes that the service connections of the patrons should be put in at the expense of the utility, and the utility should be permitted to carry this charge as a capital charge, the same as for any other portion of the property. It is unfair to exclude the cost of these improvements in the valuation of the plant for rate-making purposes, for, if the plant were being valued for sale, these items would enter into the valuation, and, therefore, should the sale be consummated, an injustice would be done the new purchaser, as he would not be allowed a return on these portions of the purchase valuation, which would defeat the purposes of the valuation for rate-making, namely, equity to the owner and equity to the public.

Reproduction Method of Valuation.—The writer can see no objection to the reproduction method of valuation for existing public utilities, and, for properties that have been constructed previous to

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the last decade, it is probably the only method that can be adopted satisfactorily. In the hands of engineers of experience and judicial temperament it will give a valuation equitable to all parties concerned.

The writer cannot fully agree with the report as to the propriety of deducting the depreciation on the existing plant. If the plant has earned a competent return sufficient to pay interest charge, operating expenses, and depreciation, then it is entirely proper to reduce the reproduction value of the plant by the amount of the depreciation; but, should the plant not have earned a competent return, but have gone along from year to year, paying only interest and operating expenses, it has done so at the expense of its vitality and due to its substantial construction. The public has received the benefit in low rates due to the plant's vitality, and therefore it is only just to the owner that he should be allowed to receive an additional return, which, on an amortization basis, would cover the accrued depreciation, at the time of valuation, at the expiration of the normal life of the plant. It is realized that this is not placing the burden on those who should have borne it, but it is the best that can be done; otherwise, the owner is compelled to put new capital into the plant to meet the demands of the public, or to suffer a total loss of that portion of his capital, or be willing to accept a return that is not a reasonable one. We are justified in placing this additional burden on the present and future generations by common consent; witness the issuing of long-term bonds for public improvements, etc.

Unit Prices.—The use of the average prices of material for periods of 5 or 10 years, ordinarily speaking, is fair, but occasionally there are times when this method would do an injustice to one of the parties. For instance, during the past few years, the price of cast-iron pipe has been as high as $36 per ton, and it is now approximately $24 per ton, a reduction of 25 per cent. A water-works property properly designed and constructed in 1906 and 1907, when pipe was selling at from $30 to $36 per ton, is just as capable of performing service as if constructed to-day; therefore, to value it on the price of pipe since 1907 would be an injustice to the water company, and then again, a plant constructed in 1898 when cast-iron pipe was down to $18 per ton, valued in 1906 at $30 per ton would be unjust to the public, so it is again seen that the judgment and discretion of the appraising board must be used if equity between all parties is to be obtained.

The writer believes that the prevailing rates for labor, during the previous 12 months, in the vicinity of the works that are being valued, should be used, as the rate of wages for labor is constantly increasing and will probably never be less than it is at present.

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Overhead Charges.—The writer endorses fully the report with reference to the items of overhead charges, as with few exceptions, they are probably universally under-estimated. Preliminary promotion and engineering expenses are in a great many instances paid out of other resources than those of the company, and, due to the optimism of the engineers, the preliminary and organization expenses are grossly under-estimated. The writer ventures to state that very few engineers realize the actual cost of reproducing their plans and blue prints covering any extensive work. Their cost records in this department of their work are with few exceptions neglected.

Development Expenses.—In this the Committee gives a new term for an old one without aiding or elucidating the difficulties that have been met with in the use of the old term. The discussion of "going value", as applied to a utility, has probably covered more space in the proceedings of the various technical societies than any other single subject in recent years. Some claim that it is nothing but a guess; others that it is another way of expressing the deficits of the company that have accrued from the time of its completion to the date of the valuation. Charles Gobrecht Darrach, M. Am. Soc. C. E., argues that "going value" is a misnomer, and that the proper term should be adolescence, comparing it to the growth and development of a child from birth to manhood. Undoubtedly, an operating plant has a greater value than a non-operating plant, and whatever this value may be called, the utility is entitled to earn a return on it, as it represents an expenditure of capital or its equivalent. Whatever the term used for this value, the writer would define it as that value of an existing operating plant having consumers and revenue over and above the value of a similar plant in the same locality having the ability to serve but no consumers or revenue. This value is independent of whether or not the utility is earning a satisfactory return. It is a value, however, that is susceptible of quite a bit of variation, but if approached with an open and judicial mind, by appraisers of skill and experience in construction and operation, it will give equitable results to all parties.

Depreciation.—The writer regrets that the Committee finds it necessary to offer a new name for the depreciation allowance. It offers the new term, "Equal-Annual-Payment Method", which is just as difficult to explain as the sinking-fund method, and, if not read carefully, gives one the impression that it proposes to repay to the investor a portion of his capital annually and thereby reduce his interest return. The equal-annual-payment method is substantially the same as the sinking-fund method, except that the latter requires that the annual amount of depreciation be used in the business, and therefore, the writer would much prefer the term "Uniform-Investment-Charge

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Method", used by Mr. Robert H. Whitten.* By this method the amount of the depreciation is put back into the works and bears the same return as the other portions of the capital, and the advantages are that it insures to the investor a constant one hundred cents on the dollar value in the property and at the same time permits the public to obtain a minimum charge for service rendered, as the amount of the depreciation of the original cost is reinvested in the property and is made to earn the same interest charge as the original capital.

The amount of this depreciation, however determined, should not be deducted from the value of the plant when fixing the rate of return to be allowed the utility, but only considered in determining the additional return to be allowed to cover the wear and tear, obsolescence, and inadequacy. It is obvious that, should the utilities be allowed only a return on the depreciated value of their properties, they could not meet their interest charges after the first year.

The writer's objection to any of these methods is the multiplication of terms and the difficulty in explaining to the investing public, and possibly the banker, the method proposed to be followed in allowing for depreciation, whereas the sinking-fund method is generally understood. The results obtained by either method are identical, provided the depreciation charge is put into the business and not returned to the original investor, or invested in outside securities.

The term "Uniform-Investment-Charge Method" by its name conveys the idea that the investment is maintained at a uniform value and that the charge to the rate-payers is uniform.

Appreciation.—The writer agrees with the Committee with reference to appreciation, notwithstanding the fact that quite a few recent Court decisions have viewed it differently. Certainly, if a utility asks for a return to meet depreciation, it is only fair to the public to grant them a return for any appreciation in the value of its works due to the growth of the community, etc.

Land Valuing.—Any engineer of experience in the promotion and construction of public utilities is fully aware of the appreciation in land value due to the knowledge that it is to be used by a public corporation. The owner of land, with few exceptions, on learning that his property is required by a utility or corporation, at once places an inflated price on it. Further, the prospective utility can afford to pay almost any price rather than proceed by law to condemn under the provisions of eminent domain, as the general public views all utilities as legitimate spoils. Of course, an owner of property is entitled to a reasonable valuation for the property taken, together with the consequential and severance damages, but, in the majority of cases, boards of jurors have taken a too liberal view and awarded inordinate damages. Therefore, a board appraising public utilities

*Ably set forth in *Engineering News* May 8th, 1913.

should be equally liberal in valuing the property of the utility, and for this purpose should consult with and obtain the views of real estate men having to do with the purchase of properties for the class of utilities being appraised. Mr. Gibson.

Rates Based on Physical Value Only.—The writer heartily endorses the opinion of the Committee that the rates should be based on other items than the bare physical value of the plant, and, in this line, thinks that it should certainly cover the rates on such intangible values as strategic location, efficiency, etc. He, however, does not believe that, because a utility has earned exceptional rates, due to its non-regulation by utility committees, it should continue to do so in the future, although justice to the innocent investors would require that their rates be reduced, not at one sweep, but rather on a sliding scale, so as to allow a sufficient time for readjustment.

Continuous Regulation by Commissions.—The writer agrees with the Committee that the regulation should be continuous and not spasmodic, but it absolutely fails to touch on the point of how often the regulation and readjustment of rates should be undertaken. In justice to the public and the investor, the rates should be adjusted to cover as long an interval as possible, and the writer had hoped that the Committee would make some recommendation as to the desirable interval between adjustments, as at present the companies can be made to defend themselves yearly before the commission. This means unnecessary expense and harassment to the company, deters public investors, and accomplishes no beneficial results. The Committee has not covered the question of the treatment of the expenses of financing, possibly thinking it was not necessary to do so. Neither does it discuss the question of intangible assets.

It has been generally decided that discount on bonds should be amortized. Therefore, in figuring the returns on the property, the interest rate should be increased to permit of this amortization fund. There is certain intangible property, which should also be amortized—such items as legal expenses, preliminary engineering, and promotion—for all these expenses ceased to exist at the expiration of the physical life of the plant, and a sufficient return should be earned so as to enable the utility to write these charges off its books at the expiration of a certain interval of time.

The Committee does not touch on the phase of the publicly owned utility, but seems to confine itself entirely to that of the privately owned. With few exceptions, the present laws of the various States have exempted the publicly owned utilities from regulation. This does a great injustice to the private utility. The latter is, to say the least, the agent of the State as much as the public utility, and therefore, though a publicly owned utility should not be permitted to earn a profit, it should be made to earn rates sufficient to cover

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operating expenses, interest, and depreciation, and not be allowed to make up its deficits out of the general tax fund. If this had been made compulsory in the past, the public would more readily recognize the benefits that have been conferred on it by the privately owned utility.

Public utilities, by their immobility and the nature of their business, must of necessity be monopolies, and therefore, to permit duplication of utilities is the grossest kind of economical waste, and should not be permitted under any circumstances.

In conclusion, the writer had hoped that the Committee would adopt a definite nomenclature for the various terms used in valuations, giving clear and concise definitions of the terms used. This is one of the first problems that should be undertaken in formulating and standardizing a method of procedure on so great a subject as this.

Mr. Vensano.

H. C. VENSANO, ASSOC. M. AM. SOC. C. E. (by letter).—This report has been read with great interest. It should be of great value to all interested, and seems to go very thoroughly and clearly into all parts of the question. There is still, however, in the writer's mind, some doubt as to the proper method of handling depreciation.

It would seem to the writer, after going over this matter, that all methods thus far suggested become practically one after a number of years of application to a particular property. The only difference seems to be in the amount finally arrived at as the depreciation increment.

In substantiation of this, two tables are submitted: Table 1 is drawn up to correspond to the table on page 34 of the report, the only difference being that Table 1 has been made up on the basis of a property having an assumed life of 5 years. This is given for purposes of comparison with Table 2. A property having a life of 5 years has been used for simplicity and in order to shorten the calculations. Table 2 considers a complex property in which the annual depreciation increment can be at once reinvested. The left half of this table shows the remaining value of the property from year to year on which it is allowed to earn an investment return. The right half shows the depreciation increments reinvested each year in accordance with the Committee's suggestion that such "depreciation allowance" should be "treated as a part of capital and used for any purposes for which the corporation is authorized to use capital". Each column in turn then shows the depreciation allowances on the reinvestment of the preceding year's allowance. In the last column of the left half of Table 2 it is shown that the total value of the property on which interest is allowed to be earned has been maintained at $100, or its full original value, so that $5 is constantly being earned as an investment return.

In the final column on the right of Table 2 is shown the total depreciation allowance for each year. Mr. Vensano.

TABLE 1.—ILLUSTRATION OF EQUAL-ANNUAL-PAYMENT METHOD OF COMPUTING DEPRECIATION.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Age, in years.	Value at end of year.	Depreciation during year.	Return on Remaining Value of Property at:		Combined Depreciation and Return on Investment at:	
			5%	7%	5%	7%
0	$100.00					
		18.0975	$5.0000	$7.0000	$23.0975	$25.0975
1	81.9025					
		19.0023	4.0952	5.7332	23.0975	24.7355
2	62.9002					
		19.9525	3.1450	4.4030	23.0975	24.3555
3	42.9477					
		20.9501	2.1474	3.0063	23.0975	23.9564
4	21.9976					
		21.9976	1.0999	1.5398	23.0975	23.5374
5	0.00					
		$100.0000				

Assumptions: Property having 5-year life valued when new at $100. Computations of depreciation allowances based on 5% interest compounded annually. Annual return on capital invested at 5 and 7 per cent.

Particular attention is called to this right hand column, from which it can be seen that at about the fifteenth year's life of a property, the component parts of which have an average 5-year life, the annual depreciation allowance, under the equal-annual-payment method, so-called, has become constant and practically remains so during succeeding years. In other words, both investment return and depreciation allowance have become constant and we have arrived automatically at a straight-line or railroad method. The amount arrived at, of course, is different, but the method has automatically become the same. It is to be noted, also, that this is absolutely the same result that would be arrived at by following out the sinking-fund method, where the sinking fund has been reinvested in the property, so that equal rates of return have been obtained on the sinking fund and the original property.

In Table 2 the same rate of return, namely, 5%, has been allowed on both the original investment and in figuring the sinking-fund increment, and the writer does not agree with the Committee in advising that these rates be different, because it is apparent that, by reinvesting a sinking-fund increment in the property itself, the same rate of return as on the original property can be obtained.

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Return on value of investment at 5%.	Total value of investment.	Value of Capital Reinvested.														
		16th year.	15th year.	14th year.	13th year.	12th year.	11th year.	10th year.	9th year.	8th year.	7th year.	6th year.	5th year.	4th year.	3d year.	2d year.
$5	$100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	22.27
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	27.42	18.24
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	33.75	22.48	14.00
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	41.56	27.63	17.26	9.56
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	28.06	34.05	21.21	11.78	4.90
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	30.35	22.97	26.15	14.48	6.03	.....
5	100	.....	.....	.....	.....	.....	.....	.....	.....	32.21	24.87	17.64	17.85	7.42	.....	.....
5	100	.....	.....	.....	.....	.....	.....	.....	33.33	26.40	19.10	12.04	9.14	.....	.....	.....
5	100	.....	.....	.....	.....	.....	.....	33.22	27.29	20.28	13.04	6.17	.....	.....	.....	.....
5	100	.....	.....	.....	.....	.....	31.31	27.22	20.96	13.85	6.68	.....	.....	.....	.....	.....
5	100	.....	.....	.....	.....	32.07	25.65	20.90	14.31	7.10	.....	.....	.....	.....	.....	.....
5	100	.....	.....	.....	32.47	26.27	19.70	14.27	7.33	.....	.....	.....	.....	.....	.....	.....
5	100	.....	.....	32.51	26.58	20.17	13.45	7.31	.....	.....	.....	.....	.....	.....	.....	.....
5	100	.....	32.32	26.62	20.41	13.77	6.89	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	100	32.11	26.47	20.44	13.93	7.05	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

ASSUMPTIONS:

Property, 5-year life.
Value of property when new — $100.
Depreciation allowance based upon 5 per cent.
Interest compounded annually.
Annual return on capital invested, 5 per cent.

Under Government regulation and fair adjustment of rates, such reinvestment should presumably be entirely safe, and should, therefore, without question, be entirely permissible.

In explanation of the construction of Table 2, it might be well to state that Column 22 is the depreciation return from year to year on the original investment. Column 23 assumes the reinvestment of the $18.10, which was the depreciation return for the first year, and shows the annual depreciation return on this $18.10. Column 24 then, similarly, shows the depreciation allowance on $19 (being the depreciation allowance on the original investment for the second year), plus $3.27 (being the depreciation allowance on the reinvestment of the first year, $18.10). Each column in turn then represents depreciation allowances on a reinvestment of all depreciation allowance accrued during the preceding year. The left side of the table, similarly, shows the remaining values for each year's reinvestment. These computations have been made with the slide-rule, and the sum of the items may not quite check with the required totals.

The writer feels that some confusion has obtained in the past in the handling of suggested methods of depreciation allowance because of

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1st year.	Value of original property.	Age, in years.	Depreciation of original property during year.	Depreciation Allowance on Preceding Years Reinvested.															Total yearly depreciation.
				1st year.	2d year.	3d year.	4th year.	5th year.	6th year.	7th year.	8th year.	9th year.	10th year.	11th year.	12th year.	13th year.	14th year.	15th year.	16th year.	
.....	100.00	0	.....																
18.10	81.90	1	18.10																18.10
14.83	62.90	2	19.00	3.27															22.27
11.38	42.95	3	19.95	3.44	4.03														27.42
7.77	22.00	4	20.95	3.61	4.23	4.96													33.75
3.98	0	5	22.00	3.79	4.44	5.22	6.11												41.56
.....	.....	6	.....	3.98	4.66	5.48	6.42	7.52											28.06
.....	.....	7	.....	.....	4.90	5.75	6.73	7.90	5.07										30.35
.....	.....	8	.....	.....	.....	6.03	7.06	8.30	5.32	5.49									32.21
.....	.....	9	.....	.....	.....	.....	7.42	8.71	5.60	5.77	5.83								33.23
.....	.....	10	.....	.....	.....	.....	.....	9.14	5.87	6.06	6.12	6.03							33.22
.....	.....	11	.....	.....	.....	.....	.....	.....	6.17	6.36	6.43	6.33	6.02						31.31
.....	.....	12	.....	.....	.....	.....	.....	.....	.....	6.68	6.75	6.65	6.32	5.67					32.07
.....	.....	13	.....	.....	.....	.....	.....	.....	.....	.....	7.10	6.98	6.63	5.95	5.81				32.47
.....	.....	14	.....	.....	.....	.....	.....	.....	.....	.....	.....	7.33	6.96	6.25	6.10	5.87			32.51
.....	.....	15	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	7.31	6.56	6.40	6.17	5.88		32.32
.....	.....	16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	6.89	6.72	6.48	6.18	5.85	32.12

the apparent endeavor to fit them to the "necessary monetary reinvestment for depreciation" rather than to the "actual depreciation" itself. He believes that there should be a clear distinction between these two, depreciation being thought of rather as the actual inherent curtailment of life of the various parts of the property, whereas the monetary replacement requirements may be something very different.

In general, in a large and complex property, the true depreciation is very nearly a straight line, whereas the monetary requirements for replacement of its component parts may vary greatly. In general, for the first few years of life, such requirements would seem to increase from little or nothing upward, and gradually approach an average, although annually fluctuating above and below such average.

For instance, assuming a property in which the longest-lived single item would have a life of 10 years and would have other items of lives of 1, 2, 3 years, etc., up to 10 years. At the end of the first year the depreciation on the item of 1 year's life would be 100%; on the item of 2 years' life, 50%; on the item of 3 years' life, 33⅓%, etc. At the end of this year the item of 1 year's life, of course, would have to be replaced, and at the end of the second year we would again have 100% depreciation for the item of 1 year's life, 50% for the item of 2 years' life, 33⅓% for the item of 3 years' life, etc. At the end of this year the items of 1 and 2 years' life would have to be replaced, and would be so replaced. Thus, at the end of the third year we

Mr. Vensano. would again have the actual depreciation for each class of items as before. Now, note the variation in replacement requirements. At the end of the first year we must replace all the items of 1 year's life; at the end of the second year all the items of 1 and 2 years' life; at the end of the third year all the items of 1 and 3 years' life; at the end of the fourth year all the items of 1, 2 and 4 years' life; at the end of the seventh year all the items of 1 and 7 years' life.

For properties having parts of extreme lives, say 100 years, we might have replacement in the eightieth year, for instance, of properties of 1, 2, 4, 5, 8, 10, 16, 20, 40 and 80 years' life; whereas, in the seventy-first year, for instance, we would only have to replace the items of 1 and 71 years' life, showing a great range in the replacement requirements.

It might seem, therefore, that, considering the "actual depreciation", the straight-line method might work out most simply and still be entirely logical, the only objection being that, for the first few years of the property's life, a too high depreciation allowance would have been made to cover "monetary replacement requirements", but, as the property ages, the results obtained would be very nearly correct, as shown by Table 2.

Although making this statement, the writer still believes that the sinking-fund method is the preferable one, and should be used. Of course, he realizes that, because of Court rulings, rates must be fixed on depreciated property values, and the use of this method becomes somewhat difficult, but he thinks that by proper presentation of cases in future such decisions might be reversed and it might be allowed (where properties have been properly handled) to base figures for rate-making purposes on the undepreciated value of the original properties, and thus make this method usable.

The Committee's suggested method gives the same results, as previously stated, as the sinking-fund method, and the Committee has no doubt met the Court requirements, but, as just shown, it would seem that the same result could be arrived at by the straight-line method with less complications in the figuring of the depreciation allowances, and with a very material simplification of corporation accounting, which it would seem would become very complicated under the suggested method.

Mr. Crehore. WILLIAM W. CREHORE, M. AM. SOC. C. E. (by letter).—The writer is in accord with the Committee's views on all the important features of this report; the only suggestions he has to offer relate to matters of, perhaps, minor significance; but which, if modified, might be a greater force in convincing those who are opposed to the principles and methods outlined by the Committee.

Mr. Crehore.

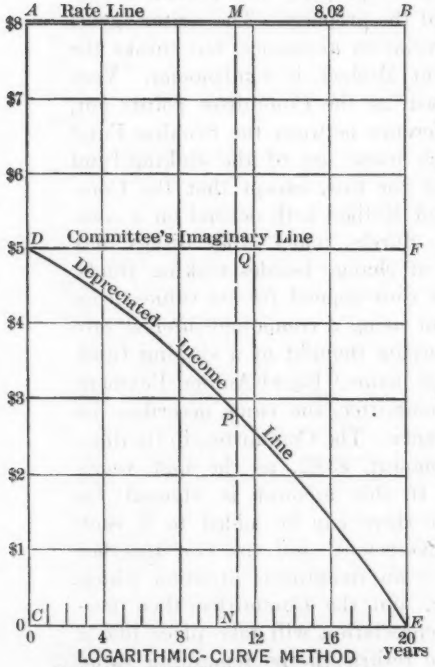
Choice of Name.—As was well known, the real determination of the rate involves the whole theory by which a depreciation allowance is to be made. The Committee advocates what it calls the Equal-Annual-Payment Method, asserting that by its use the same results will be obtained as by the so-called Sinking-Fund Method, and then proceeds to show that, if depreciation takes place at a 5% compound-interest rate, the sum of the allowed depreciation during any one year and a 5% return on the remaining value of the property will provide a constant rate throughout the whole life of the property. The writer agrees with the Committee's theory of depreciation allowance, but thinks the adopted name, Equal-Annual-Payment Method, is a misnomer. Very great confusion has arisen in the past, as the Committee points out, from a misunderstanding of the difference between the Sinking-Fund Method, as such, and methods which make use of the sinking-fund curve. There seems to be no reason for this, except that the Committee's method and the Sinking-Fund Method both depend on a compound-interest rate of depreciation. Surely, many other things may depend on a compound-interest rate of change besides sinking funds, and just because a sinking fund does thus depend for its values from time to time is no good reason for not using a compound-interest rate when desirable, without the accompanying thought of a sinking fund.

Another reason for rejecting the name, Equal-Annual-Payment Method, is that, as applied by the Committee, the name describes the the rate but not the depreciation allowance. The Committee, in its illustration on page 34, computes an amount, $3.02, as the first year's depreciation, and then states that if this amount is allowed for depreciation in each succeeding year there can be added to it each year 5% interest on the original investment, and the two together will provide a constant rate. This is an imaginary situation which will never exist after the first year, and the Committee then proceeds to explain that, although the depreciation will take place like a compound-interest calculation, yet the return on the remaining value of the property will follow it on down, so that the sum total will always be a constant rate. According to this, then, the compound-interest curve is the real theoretical depreciation line, and not the Equal-Annual-Payment line at all. Why, then, should not this method be called the Logarithmic-Curve Method? As the compound-interest curve is a logarithmic curve, this name will exactly describe the theory on which the depreciation is computed, whereas the Committee's name does not.

There is still another reason for getting away from the name, Equal-Annual-Payment Method. The Sinking-Fund Method calls for equal annual payments also, and thus the ambiguity arising from a confusion of the two will continue.

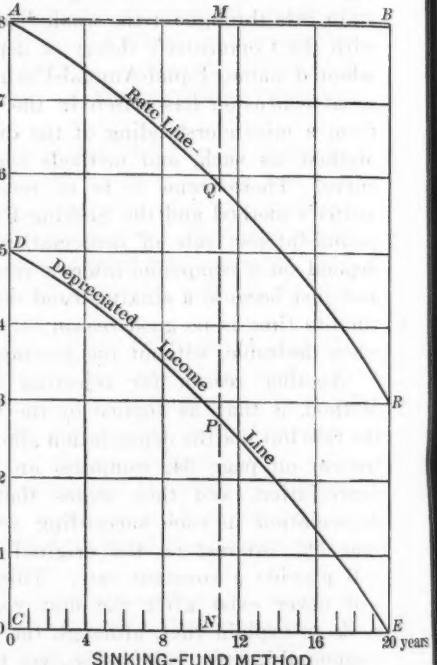
Mr. Crehore.

Logarithmic-Curve Method.—In Fig. 1 the Logarithmic-Curve Method is illustrated, all values being plotted from the Committee's table on page 34. The total rate, which is constant at value \$8.02 (the top of the diagram) is made up of the two parts, one above, and the other below, the Depreciated-Income Line, *DPE*. This line follows the values given in Columns 3 and 4 of the table on page 34, so that, at any time during the life of the property, the ordinate, *MP* (above the curve), will represent the depreciation during the current



LOGARITHMIC-CURVE METHOD
SHOWING DISTRIBUTION OF THE RATE
THE COMMITTEE'S EQUAL-ANNUAL-PAYMENT METHOD

FIG. 1.



SINKING-FUND METHOD
SHOWING DISTRIBUTION OF THE RATE

FIG. 2.

year, and the ordinate, *PN* (below the curve), will represent the return on the remaining value of the property for the same year. This Logarithmic Curve, then, divides the rate at all times according to the theory advanced by the Committee for the treatment of depreciation. The Committee's imaginary line of equal annual payment is seen at *DF*, running horizontally.

Sinking-Fund Method.—The statement that the recommended method gives the same results as the so-called Sinking-Fund Method is only partly true so far as only the items of annual depreciation

Mr. Crehore.

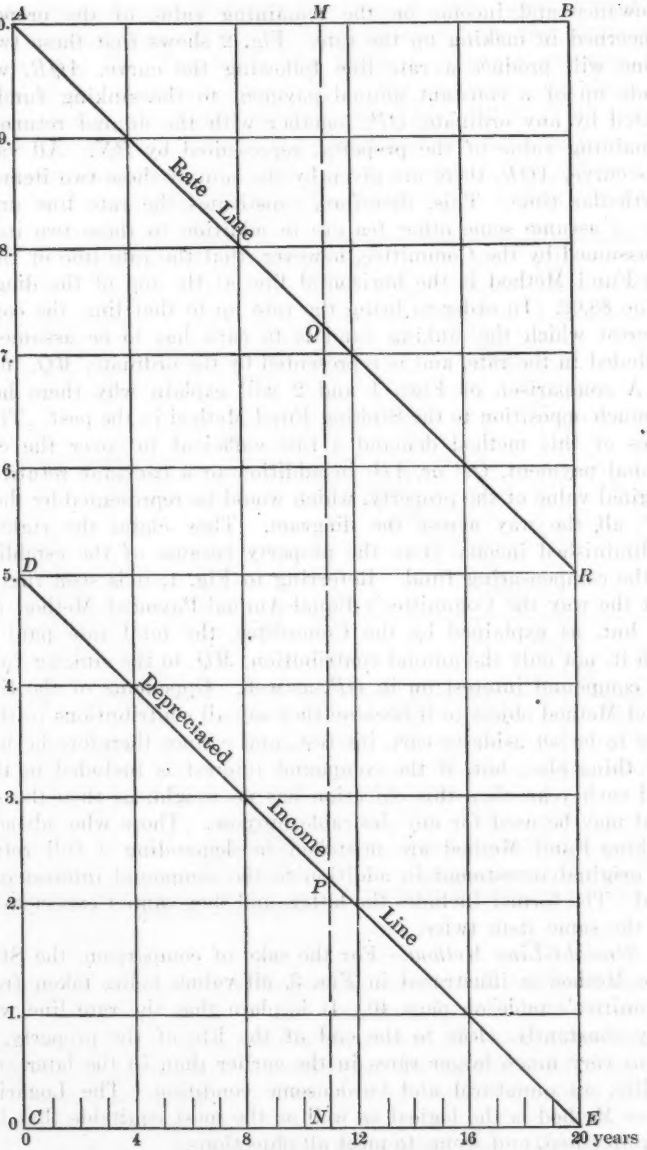
allowance and income on the remaining value of the property are concerned in making up the rate. Fig. 2 shows that these two items alone will produce a rate line following the curve, *AQR*, which is made up of a constant annual payment to the sinking fund, represented by any ordinate, *QP*, together with the annual return on the remaining value of the property, represented by *PN*. All values on this curve, *AQR*, then, are given by the sum of these two items at any particular time. This, therefore, constitutes the rate line unless we are to assume some other feature in addition to these two items. It is assumed by the Committee, however, that the rate line of the Sinking-Fund Method is the horizontal line at the top of the diagram at value $8.02. In order to bring the rate up to that line, the compound interest which the sinking fund is to earn has to be assumed to be included in the rate, and is represented by the ordinate, *MQ*, in Fig. 2.

A comparison of Figs. 1 and 2 will explain why there has been so much opposition to the Sinking-Fund Method in the past. The advocates of this method demand a rate sufficient to cover the constant annual payment, *QP* or *AD*, in addition to a constant return on the original value of the property, which would be represented by the value, *DC*, all the way across the diagram. They claim the right to an undiminished income from the property because of the establishment of the compensating fund. Referring to Fig. 1, it is seen that this is just the way the Committee's Equal-Annual-Payment Method is made up, but, as explained by the Committee, the total rate paid carries with it, not only the annual contribution, *MQ*, to the sinking fund, but the compound interest on it, *QP*, as well. Opponents of the Sinking-Fund Method object to it because they say all contributions to the fund have to be set aside to earn interest, and cannot therefore be used for any thing else; but, if the compound interest is included in the rate paid each year, then this objection has no weight, as then the sinking fund may be used for any desirable purpose. Those who advocate the Sinking-Fund Method are mistaken in demanding a full return on the original investment in addition to the compound interest on their fund. The former includes the latter, and they cannot expect to charge for the same item twice.

Straight-Line Method.—For the sake of comparison, the Straight-Line Method is illustrated in Fig. 3, all values being taken from the Committee's table on page 40. It is plain that the rate line will fall away constantly, clear to the end of the life of the property, which means very much larger rates in the earlier than in the later years of its life, an unnatural and burdensome condition. The Logarithmic-Curve Method is the logical as well as the most equitable that has yet been proposed, and seems to meet all objections.

Logarithmic Curve of Depreciation.—In Fig. 4 is illustrated the curve of depreciation itself, based on the Logarithmic-Curve Method.

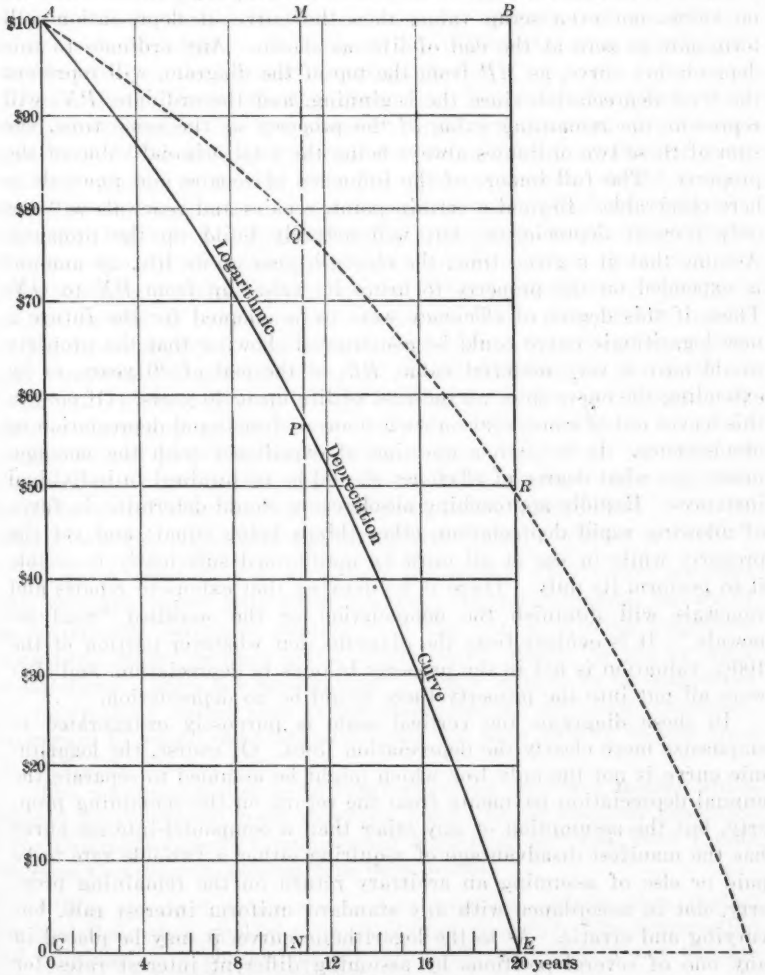
Mr. Crehore.



STRAIGHT-LINE METHOD
SHOWING DISTRIBUTION OF THE RATE

FIG. 3.

Mr. Crehore.



LOGARITHMIC-CURVE METHOD
SHOWING ASSUMED DEPRECIATION

FIG. 4.

Mr. Crehore.

The upper horizontal line of the diagram, *AB*, represents the full value of the property at which the depreciation curve starts in the first year. If it be assumed that at the end of the property's life it will have no value, not even scrap value, then the curve of depreciation will terminate at zero at the end of life, as shown. Any ordinate to this depreciation curve, as *MP* from the top of the diagram, will represent the total depreciation since the beginning, and the ordinate, *PN*, will represent the remaining value of the property at the same time, the sum of these two ordinates always being the total original value of the property. The full import of the influence of repairs and renewals is here observable. Beyond a certain point, repairs and renewals will not only prevent depreciation, but will actually build up the property. Assume that at a given time, the eleventh year of its life, an amount is expended on the property to bring its value up from *PN* to *QN*. Then, if this degree of efficiency were to be adopted for the future a new logarithmic curve could be constructed showing that the property would have a very material value, *RE*, at the end of 20 years, or by extending the curve show an increase of life up to 30 years. Of course, this leaves out of consideration such items as functional depreciation or obsolescence. It is often a question of expediency with the management, just what degree of efficiency should be maintained in individual instances. Rapidly approaching obsolescence would determine in favor of allowing rapid depreciation, other things being equal; and yet the property while in use at all must be maintained sufficiently to enable it to perform its duty. There is no denying that extensive repairs and renewals will diminish the depreciation or the so-called "final renewals." It is evident from the diagram that whatever portion of the 100% valuation is not in the property belongs to depreciation, and if it were all put into the property there would be no depreciation.

In these diagrams the vertical scale is purposely exaggerated to emphasize more clearly the depreciation lines. Of course, the logarithmic curve is not the only line which might be assumed to separate the annual depreciation payments from the return on the remaining property, but the assumption of any other than a compound-interest curve has the manifest disadvantage of requiring either a variable rate to be paid or else of assuming an arbitrary return on the remaining property, not in accordance with any standard uniform interest rate, but varying and erratic. As to the logarithmic curve, it may be placed in any one of several positions by assuming different interest rates for each position, the position of no interest rate being of course the straight line itself. A curve of this sort is preferable to a straight line, for it is well known that most kinds of property depreciate more rapidly toward the end of life than near the beginning. The determination of the correct position of this line for a given item of property can and should be approximated by appraisals taken from time to time

or by knowledge previously gained from some similar item of property. Opponents of all theoretical methods of depreciation analysis deny their practicability, and claim that no theory can be made to govern the matter. It should be the endeavor of advocates of theoretical depreciation to adopt such a theory as will conform as closely as possible with practical results by appraisement. Mr. Crehore.

Payment of Depreciation a Return of Capital.—The Committee takes the strong position that these principal and interest payments, as depreciation allowances, are actual returns of capital to the corporation, and should be treated as such. These payments should be expended by the corporation only as capital is spent. Failure to observe this point has been the cause of many a financial catastrophe. Those having authority over public utilities should insist on this point, in order to prevent misleading the owners of the property as to their real income, as well as to protect the rate-payers from subsequent demands on them to supply, a second time, depreciation contributions which have previously found their way to the stockholders in the guise of dividends. If these payments have been properly reinvested as capital, then an inventory would be sure to include them, although the particular item of property, on which this depreciation money was paid to the corporation, might be appraised at its depreciated value. There could be no injustice in this transfer of capital expenditure from one asset to another. The injury always arises from the misuse of capital.

C. P. Howard, M. Am. Soc. C. E. (by letter).—The writer has read this very able report with great interest. As stated therein, the result is "a physical valuation, dealing with the cost of existing structures with little regard to their merit or efficiency", which, "is not by itself a satisfactory basis for rate-making". Other matters to be considered by rate-making bodies, such as favorable location, are briefly discussed, in regard to which the Committee "has not been able up to this time to formulate general rules". Mr. Howard.

The recommendation as to appreciation of land is a matter which should be considered most carefully before being accepted generally. The Committee quotes Commissioner Maltbie of the New York Public Service Commission, closing with these words:

"'Why should these matters be considered less definite when applied to land than when applied to the buildings thereon? The depreciation of buildings is a charge against operation; why should not the appreciation of land be a credit?'"

The answer to this question is obvious: The funds provided for depreciation are a part of the general expense of running the business —maintenance, repairs, and replacements. Replacements are, in a way, simply maintenance and repairs on a larger scale, and at longer intervals. Depreciation funds take care of replacements.

Mr. Howard.

The appreciation of land is entirely different. It has no necessary and direct connection with operation, or with the special use made of any particular piece of land. In a general way, of course, all industries in the community contribute to it. All other owners of property enjoy this increase as a clear bonus; the unearned increment. Why should the investor in a railroad be selected as the one of all others who shall not receive it? It is proposed to take from the corporation this benefit.

If the land increases in value, the rates to the public are to be correspondingly reduced.

The citizens of the community will enjoy, not only the increase in value of their own lands, which increase was created in part by the railroad, but also any increase in the value of the railroad lands, which sum in effect is to be taken from the company and distributed among them.

The man who buys a farm reaps his crops, and takes the profits. Any general increase in the value of land is his also. Why should it be different with a railroad company?

The Committee quotes from the decision of the United States Supreme Court in the Consolidated Gas Company case, in part as follows:

"'If the property which legally enters into the consideration of the question of rates has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule.'"

If the corporation shall "in effect" pay "to the public the amount of the appreciation" as recommended by the Committee, what benefit will the company receive from such increase? Absolutely none.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

DISCUSSION ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.*

BY MESSRS. W. H. FULWEILER, SANFORD E. THOMPSON, AND PRÉVOST HUBBARD.

Mr. Fulweiler.

W. H. FULWEILER, ASSOC. M. AM. SOC. C. E. (by letter).—The writer regrets his inability to be present to congratulate the Committee on the subject-matter and method by which it has collected and crystallized into concrete form so much of the recent development in the use of bituminous materials.

The use of such materials has advanced so rapidly during the past few years that naturally differences of opinion have arisen regarding the many details that enter into their utilization in road construction and maintenance. Many of these points have been covered by the Committee in such a manner that the writer believes that the report will be of the greatest practical benefit to many engineers, who, though they have been in charge of road construction, have been unable, through press of other duties, to give this subject the attention that it deserves, and who, therefore, have been frequently led into erroneous opinions. Relative to this matter special attention should be called to the Committee's statement regarding its conviction that there is no such thing as a "Panacea" for all highway ills, as the writer believes that at the present time there is a tendency on the part of some road authorities to insist on the use of a single type of construction under any and all conditions, which tendency has failed, and will fail, in many cases to result in the most economic type of construction.

*This is a discussion on the Report of the Special Committee on Bituminous Materials for Road Construction, for 1913, presented at the Annual Meeting, January 21st, 1914.

Mr. Fulweiler.

In connection with the use of the traffic census in deciding on the most economic type of construction, it is believed that more attention should be paid to the position of the proposed improved road in relation to the general system, and the effect of its improvement on the general direction of traffic.

For a number of years the writer has held the view, as expressed by the Committee, that the objectionable slipperiness of bituminous surfaces can be decreased or prevented by proper precautions during construction, and that the succeeding paragraph in the Committee's report points out one of the principal precautions to be observed, namely, to reduce the crown on roads constructed with bituminous materials to less than ½ in. per ft.

The protection of the edges of bituminous pavements is very important, especially in view of the fact that, where there is a considerable proportion of high-speed motor traffic, many of the older roads are so narrow that they do not permit sufficient clearance without the general use of the shoulder in passing. This point, however, is already beginning to receive attention in several States.

It is believed that the Committee might have dwelt even more strongly on the fact that the character of the stone must influence the choice or grade of the bituminous material to be used, and that, after all, the stone must eventually carry the traffic, and, though cementitious qualities in the stone are no longer necessary, the other desirable qualities of hardness, toughness, and low percentage of wear, seem quite as essential as in stone for water-bound construction.

The Committee's remarks on the maximum amount of distillate up to 170° cent. allowable in tars, reminds the writer of his professor in mineralogy, who, in his lectures, would, in describing the color of a mineral, proceed somewhat as follows: "The color is generally blue, frequently green, with yellow varieties quite common. Many red specimens have been found at Franklin, N. J., and a number of mines in Colorado produce quantities of black twins, while a translucent variety is widely distributed through portions of Pennsylvania."

It is especially gratifying to the writer to note that the Committee has set forth so clearly the necessity for care in the use of bituminous materials, in the paragraph, where it is stated:

"Whatever method of construction is selected, the use of a bituminous material by no means justifies any lack of care in the ordinary details to be followed, but rather increases the need for thoroughness and skilled supervision."

The writer believes that probably 70% of the failures which have been attributed to the bituminous material have been due, as suggested by the Committee, to lack of care in details and supervision.

The writer cannot agree with the Committee in reference to the question of the use or omission of a seal coat on bituminous pavements, as he believes that the seal coat, if for no other purpose than

insurance against incidental imperfection or carelessness in the surface treatment is always advisable, and that the sole cause for its omission should be lack of funds. Mr. Fulweiler.

The writer wishes to emphasize the thought of the Committee on page 4, where it is stated that: "Where the character of the traffic justifies the use of the bituminous concrete pavement, the same conditions demand an extraordinarily strong foundation therefor." Heretofore, it is believed that too little attention has been given to the question of foundations and their effect on the life of the surface.

The writer most strongly agrees with the Committee that hand-mixing methods should be avoided where practicable, and that, in the use of a heated aggregate in the construction of a bituminous concrete pavement, non-uniformity or excess in heating the stone should also be avoided. This practically recommends the use of some mechanical form of dryer and heater, which the writer believes to be essential for uniformly successful results.

In the Construction by Penetration Method, it is believed that an equally important factor for successful results is a thorough compaction of the road metal before the application of the bituminous material, secured by proper grading of the road metal. The writer believes that compaction will never be secured with stone of uniform size until a certain proportion has been crushed into chips, which, under the action of the roller, serve to key or lock the larger pieces together, and produce that mechanical interlocking which is necessary to stability. He also believes that this same stability can be secured with very much less rolling by the use of stone of varying sizes down to, say, ¾ or ½ in., and that such a stone mixture can be compacted with very much less rolling, thus resulting in a cleaner stone surface, and, therefore, better penetration and adhesion of the bituminous material.

In Construction-Surface Treatments, with which type the writer has probably had the greatest experience, he would suggest that the first paragraph read:

"The success of surface applications is proportionate to the penetration obtained by the bituminous material into the old road surface, and that such degree of cleanliness of the latter must be had, by sweeping or other methods, as will ensure the proper degree of penetration."

As the result of his experience, which would seem to indicate that penetration into the surface of the road is essential to success, surface treatment depending entirely on surface adhesion, even if it be perfect, will disintegrate by the failure of the water-bonding of the crust beneath the bituminous treatment.

From a careful study of a large number of roads, it has seemed to the writer that the most successful results have been attained where the penetration into the surface varied between ½ in. and 1 in. A

Mr. Fulweiler.

deeper penetration (1¼ to 1½ in.) was frequently indicative of too fluid a material, with the resulting deficiency in binding or bonding strength.

Too much stress cannot be laid on the cleanliness of the surface, as this is a fundamental requirement in successful surface treatment. The writer is also quite in accord with the Committee in disapproving of the use of fine sand, and in approving of the use of clean stone chips or small gravel for surface applications, and also in strongly disapproving of any excess in material.

He does not believe that under normal conditions the application of a bituminous material on a water-bound or bituminous surface is justified, unless it is covered with a mineral aggregate, as the cost of the latter is rarely 40% of the total cost of treatment, and he believes, therefore, that the rôle of the bituminous material in maintenance work is secondary to the stone, or rather mineral aggregate, and that its primary object is to protect the stone and assist in maintaining it in such a position that it may most efficiently and economically carry the traffic.

The next paragraph he believes should have the widest publicity, especially among many engineers to whom the care of the roads is but part of their work, and that is, that "the success of results depends first upon the condition of the surface to which the treatment is applied. Even the best surface treatment cannot correct defects in the original surface, though it may reduce or modify them."

Many cases have come to the writer's attention in which an improperly constructed, water-bound road had been given a light surface treatment with the expectation that such treatment would correct all the inherent defects of the original construction.

It is believed that the results obtained during the last two or three years would have justified the Committee in expressing its opinion on pressure distribution more strongly than it has. The writer's opinion, based on some four years' work with pressure distributors, and six years' observation of gravity distribution, is that with the use of pressure distributors, from 20 to 25% of material may be saved, 50% better penetration may be secured, and the over-all efficiency, as compared with gravity distribution, is about as two is to one.

So many discussions have recently appeared in the technical journals regarding the question of definitions, that the writer is rather loath to mention the subject, but he would call attention to the Committee's definition of "free carbon," and would suggest that "organic" be inserted before "matter," as bringing this definition somewhat more in line with the methods used for the determination of this constituent. It is not believed that the use of the term "free carbon" can be defended to any very great extent, as the material is not pure carbon, and the quantity varies with the method of determination and the solvent used. The writer believes it to be better practice to

use the term "material insoluble in," or "organic material insoluble in carbon bisulphide," without attempting to give a name known to be erroneous, to a compound or mixture of varying composition. Mr. Fulweiler.

Under "water-gas tars," the writer does not believe that the word "cracking" is used correctly in this connection, but that a more correct definition would be: "tars produced by the decomposition and polarization of oil vapors in the manufacture of carburetted water gas." The same criticism might be applied to oil-gas tar.

There are several points in connection with the report which the writer would like to discuss, but which he feels he should refrain from doing, as it would involve the discussion of characteristics that serve to classify certain commercial materials, and he does not believe that such a discussion by one interested in the commercial development of bituminous materials would generally be considered as unbiased.

SANFORD E. THOMPSON, M. AM. SOC. C. E. (by letter).—In this report the writer notices that "aggregate" is defined as "The mineral portion or the larger sized particles forming the body of a concrete or of a road surfacing," etc. Mr. Thompson.

This definition is at variance with the use of the term in connection with concrete construction, and as universally adopted in practice. It also disagrees with its use by the Special Committee on Concrete and Reinforced Concrete.

The generally adopted use of the term, "aggregate," is: the entire mineral inert material, such as sand, broken stone, etc., in a concrete or a pavement.

Any other definition is illogical and unscientific, because there is no well-defined line between fine and coarse aggregates, and, in certain cases, the coarse aggregate may be a material no coarser than a very coarse sand. Even in a bituminous mixture, there is no line of demarcation after the fine and coarse materials are in place in the pavement, the effect of the mixture being simply a graded mineral substance ranging from coarse to fine. The writer, therefore, offers the following as a substitution for the definition of aggregate:

Aggregate.—The mineral inert material, such as sand, gravel, and broken stone, with which the cement or the bituminous material is mixed to form a cement concrete or a bituminous concrete. Fine aggregate may be considered as the mineral inert material finer than ¼-in. mesh in size and coarse aggregate the material coarser than ¼-in.

PRÉVOST HUBBARD, ASSOC. AM. SOC. C. E. (by letter).—The following comments are made with some reluctance on the part of the writer because for a number of years he has been engaged in an attempt to standardize the nomenclature of bituminous materials through his connection with a sub-committee of six engineers and chemists Mr. Hubbard.

Mr. Hubbard.

of the American Society for Testing Materials appointed for that specific purpose. He fully realizes the difficulties attending any attempt to standardize nomenclature, and, therefore, he is more inclined to constructive work than to purely critical work. The satisfactory definition of a single term is at best no easy matter, and when a number of terms applying to the same general subject are to be defined, the complexities increase enormously. Every term then has to be considered in its possible relation to other terms which have already been defined, and it becomes necessary to make haste slowly in order to obtain results. Definitions of a few important terms may be put forward safely at one time for discussion and criticism, and after general acceptance, they may be used as a foundation on which to build a standard nomenclature. The preparation and submission of a large number of terms simultaneously, however, is apt to cause confusion in the minds of those who have not made a study of the subject, and result in their blind acceptance of definitions which, presented singly, would have called forth considerable discussion and criticism. Also, when the definition of a term has already been adopted by a technical society of recognized standing, it would seem unfortunate, from the standpoint of standardization, for another society to accept a different definition for that term without some very good reason.

These remarks apply to certain definitions put forward by a majority of the Special Committee. The writer, therefore, takes this opportunity to discuss them briefly, and, without injecting his personal views, show why some of them, at least, should not be accepted by the Society.

The recommended definition for "Bituminous Concrete Pavement" includes "Sheet Asphalt Pavement" as also defined. This conflicts with the definition for "Bituminous Concrete," as given under the recommended definition for "Concrete". In other words, as defined, a bituminous concrete pavement might be one composed only of sand and bituminous material incorporated together. "Bituminous Concrete", however, must, according to definition, consist of an intimate mixture of gravel, shell, slag, or stone particles with certain proportions of sand and bituminous material.

The recommended definition for "Foundation" is faulty if the definition for "Roadway" is accepted. Thus "Foundation" is defined as being "the portion of the roadway below and supporting the road crust." "Roadway," according to definition, is synonymous with "Carriageway," and "Carriageway" is defined as being "that portion of the road particularly devoted to the use of vehicles." "Road" is defined as "a highway outside of built-up districts". Therefore a foundation, according to definition, occurs only as a part of a highway outside of built-up districts.

Mr. Hubbard.

In like manner the recommended definition for "Pavement" is faulty, in so far as it applies to a structure for carrying vehicular traffic. According to definition, it is first of all "the wearing surface of the carriageway" or "portion of the road" which is "a highway outside of built-up districts". A pavement, therefore, can only occur outside of built-up districts, if the recommended definitions are accepted. If these definitions are accepted, we are thus forced to the conclusion that streets, which are defined as being highways in built-up districts, cannot be pavements, and that they have no foundations.

The recommended definition for "Filler" would appear to be incomplete, inasmuch as it does not include material used to fill joints in Portland cement concrete pavements, but only in brick or block pavements.

The definition for "Bituminous Cement", if broadly interpreted, would include practically all bituminous aggregates used in highway work. This is not consistent with the customary usage of the term.

The use of the word "asphaltic", in the recommended definition for "Cut-Back Products", is either meaningless or misleading. The definition recommended in 1913 to the American Society for Testing Materials by its Committee D-4 has not been improved by inserting this word. If asphaltic is intended to mean native asphalts, its significance is not clear, and if not, it is superfluous, as many petroleum residuums are asphaltic. The term "cut-back" first of all implies previous distillation, so that a native asphalt fluxed with a distillate could hardly be called a cut-back product.

The recommended definitions for "Oil-Gas Tars" and "Water-Gas Tars" are identical with those recommended in 1913 to the American Society for Testing Materials by its Committee D-4, except that the words "high temperatures" have been omitted. It would seem preferable to make use of this qualification, as oil vapors may be cracked under a great variety of conditions, but it is only at high temperatures that cracking produces oil-tars and water-gas tars.

The recommended definition for "Free Carbon" limits the occurrence of free carbon to tars only, whereas it may occur in any type of bitumen which has been subjected to excessively high temperatures. Moreover, it is defined as "matter which is insoluble in carbon disulphide". In certain tars, such as blast-furnace tars, mineral matter may be found to the extent of 5% or more. Under the definition, this would be considered as free carbon, which, of course, is wrong. The definition adopted by the American Society for Testing Materials for "Free Carbon in Tars" is to be preferred. This definition is as follows: "Free Carbon in Tars is organic matter which is insoluble in carbon disulphide".

Mr. Hubbard.

The recommended definition for "Fixed Carbon" differs from that adopted by the American Society for Testing Materials, for no apparently good reason, and is, in fact, incorrect. The word "coke," after residual, has been left out, and the words "highly heating" have been substituted for "burning". Thus the adjective "residual" has been made to take the place of a noun. If the word "coke" is eliminated, "residual" should be changed to "residue" or "residuum". Even if this were done, however, the definition would be incorrect, as the process indicated might be conducted on a bitumen without producing fixed carbon. It is quite possible to heat asphaltic hydro-carbon products highly in the absence of free oxygen without producing coke, in which case fixed carbon would not be produced. If, however, the material is burned in the process, coke is produced and also fixed carbon.

The recommended definition for "Flux", though correct as far as it goes, does not go far enough. For example, fluxes are limited to "fluid" and "semi-fluid bitumens", whereas they may be solid products. Thus naphthalene, a white, crystalline solid, may serve as a flux for hard tar-pitch, a combination of the two producing a soft or even fluid bitumen.

In view of the foregoing it is suggested that the proposed definitions be given very careful consideration with a view of possible modification before they are accepted by the Society.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS.*

By Messrs. Edgar Marburg, P. A. Beatty, H. E. Phelps, Ernest McCullough, M. J. Henoch, and Leonard Lundgren.

Edgar Marburg, M. Am. Soc. C. E. (by letter).—The writer has read this report with interest. Inasmuch as it is the intention of the Committee to follow this with a more complete report, the writer ventures to suggest the desirability of considering the preparation of two additional average yearly compensation curves, of which one would exclude the upper and lower 5% of the returns for each group, and the other the upper and lower 10% for each group. These curves would thus represent, respectively, the average compensation of the middle 90 and 80% of the membership of each group, and, as such, would serve as more reliable criteria as to what may be called the "average salary expectation curve," according to length of professional service, in so far, at least, as such expectations can be safely premised on experiences. Mr. Marburg.

To illustrate the misleading character of the present average yearly compensation curve, attention is called, for example, to the group of 69 members who have had 27 years of experience. In that group one particular individual happens to command the extraordinary professional income of $150 000 per annum. If that individual alone were excluded from this group, the average yearly compensation of that group would be reduced by more than $2 000. In the other group, in which a single income of $150 000 appears, there are 118 members, and despite this relatively large number, the exclusion of this one individual from that group would reduce the average yearly compensation of the entire group from $5 721 to less than $4 500.

*This is a discussion on the Report of the Special Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers, presented to the Annual Meeting, January 21st, 1914.

Mr. Marburg.

The writer is strongly of the opinion, too, that the earnings of the 3 214 members who failed to respond to the circular inquiry are likely to be very much lower than those of the 3 636 respondents. The effect of the exclusion of this silent element on the returns reported by the Committee must necessarily remain a matter of conjecture, but, in the writer's judgment, it can hardly be doubted that its effect, if it could be determined, would be decidedly in the direction indicated. It is only human that an individual who realizes that he has been more or less of a failure, at least by the standard of earnings, would be reticent in his attitude toward an inquiry of this nature, notwithstanding assurances of secrecy.

The writer shares the opinion commonly held by members of the Engineering Profession, that, as a professional class, engineers are underpaid. In the broad consideration of that question, the average compensation curve is much more significant than either the maximum or minimum compensation curve. It seems especially important, therefore, that the average yearly compensation curve should also be constructed in the manner suggested, in order that the danger of drawing hasty and unfairly optimistic conclusions from the data collated by the Committee may be minimized; for even then, as has been pointed out, the showing would probably be much more favorable than is to be anticipated from complete returns from the membership of the Society, not to mention the large element—especially in the lower professional ranks—which is not identified with the Society.

Mr. Beatty.

P. A. BEATTY, M. AM. SOC. C. E. (by letter).—The writer would suggest that it might be useful to prepare an "average yearly compensation" line of those who receive $15 000 per annum or less from the practice of their profession.

The percentage of engineers who receive more than $15 000 annually is very small, the writer believes, and the elimination of those who receive more would lower the "average yearly compensation" line considerably. For example, take the twenty-seventh year. The average yearly income of 69 engineers is given as $8 023. Eliminating the $15 000 man, the average annual income of 68 engineers is reduced to $5 935.

To obtain an idea of, say, 85% of the Profession, which would probably be useful in an investigation of this kind, the elimination of exceptional incomes would give a line representing the earning capacity of the greater part of those engaged therein.

Mr. Pheips.

H. E. PHELPS, ASSOC. M. AM. SOC. C. E. (by letter).—This report is most interesting and valuable. However, it does not seem probable that the average yearly compensation curve, as determined by the Committee, correctly represents actual conditions. The fact that only 53% of the membership of the Society reported their compensation, is

doubtless due to a variety of causes, but it seems reasonable to believe that the main cause is an unwillingness to report a low compensation. Moreover, if the words "Civil Engineers" refer—as generally assumed—to all graduates of technical schools and all others with an equivalent education and training, and not only to members of the American Society of Civil Engineers, the compensation of these men should be ascertained, if possible, and incorporated with the information already found, in order to show the true state of affairs. Mr. Phelps.

It is doubtful if the well-nigh universal dissatisfaction of engineers with their income and general status would be so pronounced if the curve presented by the Committee showed the true state of affairs, low though it is. From the writer's acquaintance among engineers, he does not believe that their average yearly compensation is one-half, or, at most, two-thirds, of that shown. Doubtless, many engineering firms are represented therein by the large incomes of the members of such firms, and not at all by the small incomes of the draftsmen, transitmen, and assistant engineers employed by them. The members of this Society should not spend too much time contemplating the dizzy altitudes reached by the maximum compensation curve, but rather should study how to raise both the true minimum and average curves, which are much too close together and lower than they should be. The Society itself should take some action along this line, and the writer believes the most effective action would be, in some way, to assist engineers to obtain employment when they need it.

ERNEST MCCULLOUGH, M. AM. SOC. C. E. (by letter).—The writer has read this report with interest, and feels certain that it will be ably discussed by the editors of the leading technical journals. He hopes that these editorials may be offered by members of the Society and be printed in the *Proceedings* and *Transactions*, for he is sure that the opinions of the editors will be a reflection of those of many members. Mr. McCullough

The report is based on replies from about one-half the membership of this Society. The total number of engineers enrolled in the four national engineering societies—Civil, Electrical, Mechanical, and Mining—cannot be more than 20 000, when one takes into consideration the number of men enrolled as members in more than one of these societies, the total of the separate lists reaching to more than 27 000. These represent the men who have achieved some measure of success, yet many among them feel that Fate has not been kind. Of the 20 000 men, less than 4 000 have answered in such a manner that their replies could be incorporated in the report.

Statistics from engineering schools show that the number of such schools has increased greatly in the past 20 years, and that of the total number of men graduated from such schools, more than 60%

Mr. McCullough.

have been graduated since 1900, just 14 years ago. The first school of engineering was established in the United States in 1824, and the first man was graduated in 1826. The increase of interest in technical education, therefore, has exceeded the increase in population, but as engineering is a profession which is peculiarly the product of civilization, the increase, perhaps, is not disproportionate to the general world advance.

Numbers of men calling themselves engineers, some of whom attain high standing, are non-graduates, and many are engaged in engineering work who did not complete a four-year course in a technical school. That numbers go into the work after partly completing a course of study in engineering schools is well known, and many of these men are good engineers. The writer has been informed by a professor that on an average probably 20% of the freshmen complete the work of the senior class. The number of alumni of a school, when compared with the graduates, shows that this statement cannot be far wrong. In a certain college in the United States—not an engineering school, however—established in 1837, the number of graduates has been 15 000, yet the living alumni number 12 000, a majority being enrolled as members of the alumni association.

This being admitted as a state of affairs existing in engineering schools, the number of men practicing engineering in the United States must be several times the number enrolled in the national societies. It may be not far out of the way to state that the less than 4 000 men whose reports are tabulated and shown on the compensation chart issued by the Committee, is a very small percentage of those whose interest in the work of such a committee is lively.

In the writer's opinion, it is not so much the matter of compensation that the Committee should consider as the conditions of employment and the treatment of the men who have received enough training in engineering to lead them to follow it as a vocation. The custom is to engage them in a hurry in order to push work to completion, and rarely is a man given a week's notice of dismissal. The majority of men employing engineers and engineering assistants prefer to keep them in ignorance as to when their services will be no longer required, fearing that they will leave for a new job before their work is completed, thus placing the employers under the necessity of getting new men and thereby increasing the cost for engineering services. It is common with railways to pay their engineers from the 20th of the month up to the 1st, thus retaining two-thirds of a month's pay. Frequently, the men are dismissed on the 1st and their pay is not forwarded to them until the regular time, giving them no surplus on which to live until it comes or a new job is secured. It is not uncommon for men to be discharged after many years of faithful service, and be replaced by younger men at lower pay.

Mr. McCullough.

The seasonal employment of engineers, so far as this factor relates to their work, puts them on a plane with common labor. The pay being intermittent and the men being subject to the same troubles that all are subject to, such as illness in their families, injuries and illness to themselves, sporadic employment at pay which is seldom more than a bare living wage when employed, being below a living wage when the average is considered, causes much needless distress in a class of men whose training must be high and who are usually well educated. The work of the engineering societies should be directed toward securing better terms for all men who have received the necessary training.

Numerous companies make a good profit in securing employment for technically trained men. Their rates are very high, and they enforce collections mercilessly. Men are given positions—which these companies say will be permanent—only to lose them in a short time, often before the commission to the employment agency has been fully paid.

The Society should start some method, in connection with the other national societies, whereby employment may be obtained, a clearing house as it were, for the placing of jobless men in touch with jobs. Members of the societies should pledge themselves to employ only men recommended to them by this department. The Committee, further, should enlarge the scope of the inquiry to cover every man who has been graduated from an engineering school within the past twenty-five years. This would imply that the inquiry into conditions existing among engineers should be extended to all engineering graduates, and not to men who are not graduates. The men, however, who have achieved such a measure of success as to enable them to be enrolled in the American Society of Civil Engineers should be treated as a distinct class. The writer knows a very large number of men fully qualified to be members of this Society who do not join for the sole reason that their income is so precarious, and who are out of work so often, that they feel that it is impossible to ally themselves with the Society. These are the men who really impress employers with the fact that many things are not as they should be with the vocation of the engineer.

Mr. Henoch.

M. J. Henoch, M. Am. Soc. C. E. (by letter).—The writer is very much interested in this report. He realizes the great amount of time and labor required to collect, compile, and digest the mass of data requisite for the proper consideration of this important subject, and it may be, therefore, that the following remarks are premature. They are not offered, however, with any intent to criticize the excellent work of the Committee, but to bring out two points which appeal to the writer.

The first is in regard to the Committee's conclusion that the membership of the Society is large enough to be representative, and to be typical of the various stages of engineering life.

Mr. Henoch.

The members of this Committee are all eminent and successful engineers, live in an environment of "big things", and very naturally view conditions from a different standpoint from those who are not so successful. The writer has no means of knowing what percentage of those actually engaged in the practice of civil engineering in the United States are connected with this Society, but the proportion certainly is small.

The membership of the Society is chosen by a process of elimination which excludes the less successful engineers, and there are thousands who, although eligible to membership under the rules of the Society, are, nevertheless, unable or unwilling to afford the expense thereof, small as it may appear to their more prosperous brethren. There are many trained and capable engineers working for salaries of from $125 to $150 per month, who have families or dependent relatives to support, and to them the payment of $25 for entrance fees and $15 for annual dues looks like an unsurmountable obstacle. It may be said, therefore, that, as a rule, it is only the more successful engineers who become members of the American Society of Civil Engineers, that the membership is not large enough to be representative, and that it is not typical of the various stages of engineering life.

Also, in regard to the curve of average yearly compensation shown on the diagram presented by the Committee: it appears to the writer that this does not show the thing which appeals to the average engineer. It is true that it shows the average yearly compensation of the engineers reporting, but it does not show the yearly compensation of the average engineer. An inspection will show that this curve is unduly influenced by the curve of maximum compensation, and that every fluctuation of this maximum curve is reflected by a corresponding fluctuation in the average curve, but the minimum curve has, in general, no corresponding effect.

It will generally be admitted that the very high salaries reported are exceptional, and are by no means as frequent as the low salaries, yet their effect on the average curve, as compared with that of the low salaries, is (roughly speaking) inversely proportional to their relative frequency. For instance, the conditions shown for 18 years' experience is an exaggerated case, but serves well as an illustration. In this case the maximum salary is 114 times the minimum, though the average is only 4.3 times the minimum. Considering the extremes only, it requires 32.74 men at the minimum salary with 1 man at the maximum salary to produce the average shown; or, in other words, it requires practically 33 men at the minimum salary to offset the effect of 1 man at the maximum salary. The ratio of the minimum to the average is 1 to 4.33, though the ratio of the average to the maximum is 1 to 26.22.

Mr. Henoch.

It would appear, therefore, that the thing which really shows the conditions of the average engineer is (to use the terms of the Committee) the salary of the "middle-man" or that compensation which is exceeded by and which exceeds that of an equal number of men.

Mr. Lundgren

LEONARD LUNDGREN, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has studied this report with interest.

The Committee states that the average curve shown in the report is, if anything, too low; that the general average of compensation of 3 636 members is $4 325; and that the average yearly compensation for 1 972 members whose experience is 15 years or less, is $2 827.

The writer feels that the impression will be created that the average engineer receives the average yearly compensation shown by the diagram accompanying the report. The initiated know that this is not the case, and that the majority of those answering the questions of the Society were the most successful engineers in the United States, which fact raised the average considerably. This point should have been emphasized in the report. If the curve was intended to show the ideal salary that the Committee thought an average engineer should be paid, it would be admirable.

It is known that the minimum wage of a technical graduate of one year's experience is $360 per year; that $720 per year is considered a good job; and that $1 200 per year is an exceptionally good position, and the writer can scarcely believe that an engineer would receive from any one the maximum salaries given in the report ($2 000 for one year's experience, and $5 000 for two), except for reasons outside of the realm of engineering.

The conditions after 20 years' experience can be analyzed similarly. The lowest wage known to the writer for this term of service is $900. An engineer is considered by his co-laborers as a success if he receives a salary of $3 000, and as very successful if he receives a salary of $5 000. The average shown by the diagram is $4 941, which is at least 50% higher than the salary of the average engineer of 20 years' experience.

The writer quite appreciates the position of the Committee as to lack of data, but he feels that it has overlooked numerous possible sources of information, such as the salary lists of the engineering branches of the United States Government, of the States and large cities, of the railroads, and of the large manufacturing corporations, many of which could have been secured by proper requests.

It was hoped that the Committee would secure sufficient information to produce a diagram, similar to that accompanying the report which, in addition to the minimum and maximum curves, would have a curve showing what salary an average engineer would secure for his working life. This it is felt has not been accomplished.

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ROAD CONSTRUCTION AND MAINTENANCE.
An Informal Discussion.*

BY MESSRS. WILLIS WHITED, WILLIAM H. CONNELL, E. W. JAMES, CHARLES J. BENNETT, A. W. DEAN, HENRY W. DURHAM, ROBERT A. MEEKER, PAUL D. SARGENT, W. W. CROSBY, P. E. GREEN, E. H. THOMES, S. WHINERY, MARK BROOKE, WILLIAM R. FARRINGTON, H. B. PULLAR, ARTHUR H. BLANCHARD, W. H. FULWEILER, AND JAMES H. STURDEVANT.

ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

Mr. Whited.

WILLIS WHITED, M. AM. SOC. C. E. (by letter).—This subject is of vital importance, because incompetency, inefficiency, or dishonesty in the handling of highway work may cause a larger percentage of waste of funds than in almost any other class of work. However excellent the personnel of the organization may be, there will be much waste unless all parties work harmoniously, steadily, and efficiently toward the end in view. There is the further difficulty, that, if the organization is not reasonably harmonious, many of the best men will become disgusted and resign, which would make it doubly hard to secure others to take their places. The subject can perhaps best be studied by taking a concrete example, such as the Pennsylvania State Highway Department.

The Department was first organized by an Act of the Legislature in 1903. The Act, however, was amended from time to time until 1911, when an entirely new one was passed, known as the "Sproul Act", reorganizing the whole Department, so that its functions may now be divided into three classes.

(1) The Department, at the expense of the State, is to improve and maintain, as funds become available, about 8 800 miles of roads, mostly

* At meetings held January 23d and 24th, 1914. As much of this discussion as possible is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Whited.

main thoroughfares, set aside by the 1911 Legislature, to which the Legislature of 1913 added about 1 200 miles.

(2) The Department, under the law, is required to furnish to counties, townships, and boroughs, as funds become available, State aid for the improvement and maintenance of highways. The State pays one-half of the expense, the other half being borne equally by the county and township, or county and borough, as the case may be, the Department having entire charge of such road work.

(3) The Department, under a Legislative Act of 1913, has technical jurisdiction over all improvements on about 82 000 miles of road in addition to the 10 000 miles previously mentioned, and these roads are under the control of the Township Road Supervisors acting in their respective townships. The Department also has charge of the purchase of road-building machinery, exceeding a certain amount. Contracts pertaining to work of this class are not valid unless approved by the Department. The State bears one-third of the expense of such improvements. This places all public roads, outside of cities and boroughs, under the more or less complete control of the Department.

For handling such extensive enterprises, involving the expenditure of many millions of dollars annually, three methods may be suggested:

(1) The appointment of a commission consisting of three or more members, who shall formulate all general policies and appoint officers to carry them out;

(2) The appointment of a single head, with advisory council;

(3) The appointment of a single head, without advisory council, except such as he chooses to accept from his subordinates for special purposes, the Legislature itself determining the general policy of the Department.

The advantage of the first method is that the united knowledge and experience of the whole commission is available for the settlement of disputed points and questions of policy. The principal disadvantages are: first, the division of responsibility; second, it is apt to become a mere debating club, which seldom accomplishes much unless dominated by one man who, being an official, can with difficulty be prevented from abuse of power. A third disadvantage of this method is that a competent commission for such a purpose is usually composed of very busy men, who have many other interests, making it extremely difficult to get them together with sufficient regularity to settle questions with the promptitude necessary for the most effective prosecution of the work, particularly as their salaries are apt to be much less than the real value of their services.

The principal advantages of the second method are that the members of an advisory council usually have an adequate salary, and can be required to hold meetings regularly. On the other hand, being

subordinates, they will have little power, and will therefore take but little interest in the work. Mr. Whited.

The third method, in which all authority is vested in a single head, constitutes in reality a despotic rule, which is by far the most efficient provided the proper man can be placed in the position and prevented from any misfeasance of office. The superiority of this form is shown by the fact that all military organizations from time immemorial have been under the control of single heads having despotic power, except that in modern warfare civil authorities have the right to deprive him of his commission in case he should abuse his powers or show incompetency. In warfare, efficiency is of paramount importance. Almost all industrial and business concerns are under the control of single heads, but with powers subject to reasonable limitations. The Pennsylvania State Highway Department is placed under a single head, whose powers are limited by the ordinary laws of the Commonwealth and by the Act under which he is appointed, which designates the policy to be pursued by the Department.

All contracts are approved first by the Highway Commissioner and then by the Governor and Attorney General. All appointments to the higher positions are approved by the Governor. All expenditures are audited and approved by the Auditor General. Such checks as these should be sufficient to prevent the head of the Department from misusing his authority in making appointments and from the misappropriation of any funds.

Under the above-mentioned Act, the Commissioner is in absolute control of the administration of his Department, assisted by two Deputy Commissioners, also under his control.

The First Deputy Commissioner, in addition to assuming the duties of the Commissioner during his absence, is in charge of the Bureau of Township Highways, and also of the granting of permits for special uses of all public roads outside of municipalities.

The Second Deputy Commissioner, in addition to acting as an assistant to the Commissioner, has charge of the licensing of motor vehicles, and the enforcement of State laws relative to the use of such vehicles on State roads. He also has charge of the finances of the Department.

The Chief Engineer has immediate technical and executive control over the whole Department excepting the Automobile Division.

There is a clerical and auditing staff at headquarters, performing the duties usual with such positions.

There is an Engineer of Bridges, who has technical charge of all bridges and culverts.

There is a Superintendent of Signs, and a Superintendent of Asphalt Construction, whose duties need no description. There is also a Division of Tests, and a Division of Statistics, charged with the duty

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of obtaining data as to the traffic on all roads under the jurisdiction of the Department.

There are fourteen Assistant Engineers, who have technical and executive control over their respective Districts, but who act only in a technical capacity on township road work, and an additional Assistant Engineer acting as office assistant to the Chief Engineer.

The maintenance of State roads is in charge of a Maintenance Engineer, subordinate to the Chief Engineer, and in charge of fifty Superintendents, each of whom has charge of the maintenance of all roads in his respective district. These Superintendents, acting under the direction of the First Deputy Commissioner, also have immediate technical charge of township roads and bridges, and are required to approve all plans for road improvements to be carried out by the Township Supervisors, of whom there are about 6 500.

All bills are approved by the various heads under whose jurisdiction they have been incurred, and are then audited by the Department Auditor and the Auditor General of the State, thus rendering impossible any considerable misappropriation of funds. All payments on contracts must be approved by the Assistant Engineer, the Chief Engineer, the Department Auditor, the Commissioner, and the Auditor General of the Commonwealth.

All surveys and plans are made by the staffs of the Assistant Engineers, and checked by the Chief Draftsman and his staff, located at headquarters. They are then approved by the proper Assistant Engineer, the Chief Engineer, and the Commissioner. Plans for bridges and culverts are made and checked in the Bridge Division, and approved by the Engineer of Bridges and the Chief Engineer.

Thus it will be seen that all matters of importance are passed upon by at least two competent men, who are entirely familiar with the conditions surrounding each case.

All contracts are approved by at least three men, in addition to the Attorney General of the Commonwealth, and all payments on contracts are checked and approved by at least three men before finally being approved by the Auditor General.

All work done under contract is under the direct supervision of an Inspector on the work, who is thoroughly conversant with the details. Such work is approved by the Assistant Engineer and inspected generally from time to time by the Asphalt Superintendent and Engineer of Bridges, each acting in his respective capacity, and is also inspected by the Chief Engineer.

Thus a serious error in design or construction, or the loss of funds through misappropriation, ignorance, or criminal waste, is effectively prevented.

Furthermore, nearly all the work is done under the eye and at the expense of the public, of whom a very considerable number are more

or less familiar with road building, and who, by appeal to the press, the Courts, or public officials, could cut short any gross incompetence, negligence, or dishonesty of employee or official. Mr. Whited.

The Act provides that funds for the improvement and maintenance of all highways, other than Township Dirt Roads, shall be distributed as equably as possible throughout the sixty-six counties of the State, which effectively prevents the Commissioner from showing any undue partiality toward any particular portion of the State. Hence there is always one man who can be held responsible for results in carrying out the orders issued by higher authority, and, at the same time, in the decision of important questions, there is always one of competent knowledge to whom he can turn for advice.

All appointments to positions in the Department are made by the Commissioner, and are not subject to other approval, except in the case of a few of the higher positions, which are subject to the approval of the Governor.

It is important that the Commissioner be given much freedom in making these appointments, because, when men having considerable authority are scattered widely over the State, absolute loyalty to the "chief" is of the utmost importance, and can be obtained more effectively by personal appointment than by any system of civil service examination.

At the same time, as many of the positions require a high degree of executive ability, as well as technical skill and knowledge, it is important that the appointments be placed, as far as possible, beyond the reach of "political spoilsmen". For this purpose it has been decided that, in the future, as far as practicable, all the higher positions shall be filled by promotion, after careful examination into the candidate's technical knowledge and experience, as well as a careful inquiry into his previous record, in order to ascertain his executive ability, trustworthiness, resourcefulness, tact, and general ability, to get the best results out of all with whom he comes in contact, whether superiors, subordinates, equals, or strangers, these qualities, at least for the higher positions, being really more essential than a high degree of technical skill or knowledge.

It is needless to add that the Commissioner himself must be possessed of a high grade of technical skill, as well as great business and executive ability, in order to accomplish in the best manner the results expected of him by those to whom he is accountable.

WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E. (by letter).—About two months ago a search for literature on "Municipal Highway Engineering Organizations" was made by the Library force of this Society. This search covered the past 5 years, and only one article dealing with this subject was found. Papers, however, have been written relating to the routine of existing highway organizations, and this has also Mr. Connell.

Mr. Connell.

been covered in some public works reports, including organization charts for highway departments, sometimes termed "street departments"; and it is apparent from some of these papers and reports that the officials in charge are making the best of the conditions under which they are operating, but that they are simply outlining and dealing with existing organizations, without comment or constructive criticism regarding the organization of a modern highway bureau. This being the case, it may be well to pause for a few moments and analyze some of the reasons for paying so little attention to the organization end of this important subject, which is second to no other municipal engineering division, from a business and engineering standpoint.

Highway engineering is not new, and considerable advance, resulting in improved pavements, has been made within the past 50 years, but this advance has been achieved by the efforts of a few engineers working under tremendous disadvantages, through lack of co-operation by the Engineering Profession and the fact that highway engineering has been considered seriously by engineers as a body only within the past few years. The highway work, in a large percentage of municipalities has been handled by all classes of officials from as many different walks of life (none of which gave them a claim to any qualification for the work), or by a city engineer who has busied himself with water, sewer, and bridge problems, the nature of which diverted his attention from highway work. The latter involves so much detail of an engineering nature that it is not apt to appeal to one who is wrapped up in the problems pertaining to other branches of the Profession. As a result, the highway work was allowed to drift along until the highway department was considered to be the property of the politician, and, still worse, only recently it has been used by some business administrations throughout the country to parcel out a few jobs to men probably deserving of some recognition; consequently, the highway bureau has been made to suffer through the appointment of men conspicuously unfit for the work. In this policy a great mistake has been made, as the highway bureau, in a large measure, is the principal show-case of the city government—the pavements representing the goods in the window—and it behooves any city or State administration, political or otherwise, to look to its highways, for the public is alive to the situation, and recent developments have proved that more people can be reached and satisfied through an engineering highway organization conducted on a high plane, than through any other branch of public work. An adequate organization, however, is essential, as a successful highway administration is dependent on conducting and controlling the work with the least friction. Ease of operation is the most important factor, and this can only be obtained through an organization commensurate with the demands on it.

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The political view of the highway department was expressed only recently by a leading political light of one of the largest cities in America, who stated that the highway inspectors in his city were all right because "they used to be truck drivers and consequently knew the streets pretty well." This statement was made in all seriousness, and to an engineer. It is small wonder, then, that many highway organizations reflect little credit on our municipalities. In fact, it is remarkable that the few engineers who have devoted their lives to this great work should have succeeded in producing so many good pavements, for in these there has been very little improvement, as far as the standard types are concerned, within the past few years, or since engineers in general have become interested in the subject.

Much has been said concerning municipal and State highway organizations by new and inexperienced officials, who, instead of employing experienced engineers to plan their organizations, attempt to do so themselves, and often write articles on the subject which only too frequently are misleading, as they are not based on good engineering judgment coupled with experience in management on a large scale, both of which are necessary in planning such organizations or writing on the subject. Of course, a man familiar with business organizations for controlling work can plan a highway or any other kind of organization, but not without the advice of one familiar with all the functions and work coming under the jurisdiction of the respective organizations. Therefore, if it becomes necessary to plan a highway organization, it is far better to have the services of an engineer versed in the principles of organization and familiar with the work. In other words, the most economical, the quickest, and the surest means of attaining the end in such an undertaking is to pick a man whose qualifications come nearest to fitting him for the work, and, in the case of highway work, select a highway engineer.

With reference to the organization itself, the writer will assume that he is dealing with a large city, and that necessary modifications should be made, depending on its size. In a small city, of course, it would not be practicable to have as many separate departments as in a large one, but the principles set forth can be followed in the handling of the work. Instead of discussing existing organizations, an attempt will be made to outline a proposed highway organization commensurate with present-day requirements. In this it is most important to determine what functions should come properly under the jurisdiction of a modern highway bureau and just where this jurisdiction should start. The first step, or the laying out of the highways, should come under the head of "city planning," which is becoming more and more a special branch of the Engineering Profession, and the work is of such a character and the field is so broad that obviously all municipal engineering departments will necessarily have to be more or less

Mr. Connell.

affiliated with the city planning division, and the head of this division might be termed the city engineer. Therefore, it will be assumed that the highway bureau, after the layout has been planned, will start with the design of all work pertaining to the highways, including parkways, park drives, and small highway bridges. The larger bridges should come under a bridge department, as bridge engineering is distinctly a special branch of the Profession. After the design, of course, all necessary engineering work relating to lines, grades, and inspection, from the grading to the finished pavement, should be under the jurisdiction of the highway bureau. Before leaving the subject of design—in view of the tendency of late in our municipalities to create a central bureau of design—it might be well to call attention to the fact that the engineers in charge of construction and maintenance of highways would necessarily become specialists in this line of work, and should control and be held responsible for the design as well as the construction and maintenance. This is simply the logical procedure which should be adopted in any such work. If, therefore, there is to be a central bureau of design—which may be very desirable—it should be under the city planning division, and the designing—as far as the construction of highways is concerned—should be supervised by the highway bureau.

The next thing to be considered is one which is becoming more serious every year, namely, sub-surface structures and encroachments, which include all underground conduits, pipe lines, vaults, etc., steps, street signs, stands, and every kind of encroachment beyond the building line. These matters, together with permits and licenses for automobiles, vehicles, etc., should be under the supervision of a branch of the highway bureau. With respect to conduits, pipe lines, etc., the only solution of the problem for the protection of the pavement, convenience of the public, and general traffic conditions, would appear to be pipe galleries; but, in large cities, which are already paved and built up, this would require an enormous outlay. This question should be considered seriously, however, in growing municipalities.

Having provided for the design, sub-surface structures, and engineering work, including inspection, the next subject is the surface of the street, and here the first thing to determine is just where the jurisdiction of this bureau should begin and end. Generally, in all municipalities, the city controls the highways from house line to house line, regardless of the division of expense for the footways, paving, repaving, etc., which varies in different cities. This is a big problem in itself, and will not be discussed at this time. Assuming, therefore, that the city has such control over the highways, encroachments, street signs, license fees for automobiles, vehicles, etc., naturally come under the jurisdiction of this bureau, and right here is a big

problem—as is also the case with sub-surface structures particularly relating to vaults—that has not been solved, namely, an equitable and just rental value to be levied by the city for all kinds of projections—underground or above the surface—beyond the house line, and license fees on all classes of automobiles, vehicles, etc. This, however, as in the case of the division of expense for footways, paving, repaving, etc., is too big a question to consider at this time. To continue with the highway organization, it will be assumed that the supervision, control, and enforcement of all the regulations governing these matters come under the jurisdiction of the bureau of highways. Mr. Connell.

The next, and probably the most important, matter to be considered in connection with the work rightfully coming under the jurisdiction of a modern highway bureau is a branch of municipal work which is closely related to highways and plays an important part in their maintenance, namely the cleaning of streets. This branch of engineering work—for it is distinctly engineering work, as the life of the pavements and roads depends to a considerable extent on the kind of cleaning they receive—has never, except in a few municipalities, been placed on an engineering basis, and in many cities this work comes under the jurisdiction of a separate department, which not only results in increased supervision and overhead charges in connection with the care of the highways, but an overlapping of jurisdiction with regard to the control. In a well-organized highway bureau, including street cleaning, the street patrol inspection can be and is taken over by the men supervising the street cleaning. This makes it unnecessary to have a separate street patrol inspection force, such as is employed in many highway bureaus. The street cleaning forces are constantly at work, covering every street in the city at frequent intervals, and, being supplied with a memorandum book or blanks for the purpose, can write down observations of the conditions of the streets as well as supervise the street cleaning work.

The tendency of the day is to standardize and control work in a manner which will not only conserve energy and do away with unnecessary duplication, as far as individual effort is concerned, but will concentrate forces so that operations can be carried on efficiently and economically along the line of least resistance, as ease of operation is one of the most important factors to be considered in planning any working organization, and no step would accomplish more in this direction than the combination of the street cleaning and highway bureaus. This would have the added advantage of placing this work more generally on an engineering basis, which would undoubtedly lead, not only to more sanitary methods of cleaning, but to methods which would be more apt to take into consideration the maintenance and care of the pavements themselves than is likely to be the case where the street cleaning is not under engineering supervision.

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In connection with the street cleaning work there is also the collection of ashes and rubbish and the collection and disposal of garbage. Although this work is not distinctly a highway matter, the collection and disposal of ashes, and rubbish particularly, is so much a factor of the problem of preventive street cleaning—in which the crusade against overloaded and leaky wagons, the throwing of papers, store sweepings, etc., into the streets, plays an important part—that it more properly belongs under the highway than any other municipal department. In a lesser degree, this is also the case with the collection and disposal of garbage. The writer, therefore, for the time being at least, or until a better solution of this problem is presented, will place both these matters under the jurisdiction of the highway bureau.

With reference to the division and subdivision of the work, assuming that all that comes under the jurisdiction of the bureau is distinctly highway work, the construction and maintenance and cleaning of streets should be under one chief engineer, with as many division and section engineers as the size of the city and the extent of the work requires. The clerical, including auditing, stenographic, and general office force, and the division of sub-surface structures and encroachments, including permits and licenses, should, through their respective heads, come under the chief engineer, as should also the division of contracts, specifications, and general office engineering. The work under these divisions should be delegated in a like manner, depending on the extent and nature of the undertakings over which the respective heads have control. Each of these subdivisions embraces so much detail, with the consequent opportunity for constructive criticism, that in the limited time assigned to this subject it would be folly to attempt to elaborate any one of them herein. Therefore only the main subdivisions and functions coming under the general principles set forth for a modern highway bureau have been taken up. The chart submitted herewith, Fig. 1, covers the organization outlined, and is intended for a city having a population of 1 500 000, and covering an area of about 130 sq. miles. The details will be left for future discussion, which must necessarily go on until modern highway engineering comes into its own.

Before closing, and in order that all may realize the magnitude of the work of highway bureaus in comparison with that of other engineering organizations, the writer calls attention, for example, to the extent of the undertaking of the Water Board of the City of New York in connection with the new water supply from the Catskill Mountains and vicinity. This work is said to be second only to the construction of the Panama Canal—the greatest piece of engineering work of the Twentieth Century. Now, no matter what may be the nature of the work requiring large expenditures, it is just as necessary to see that the money is spent judiciously and to the best advantage in one branch

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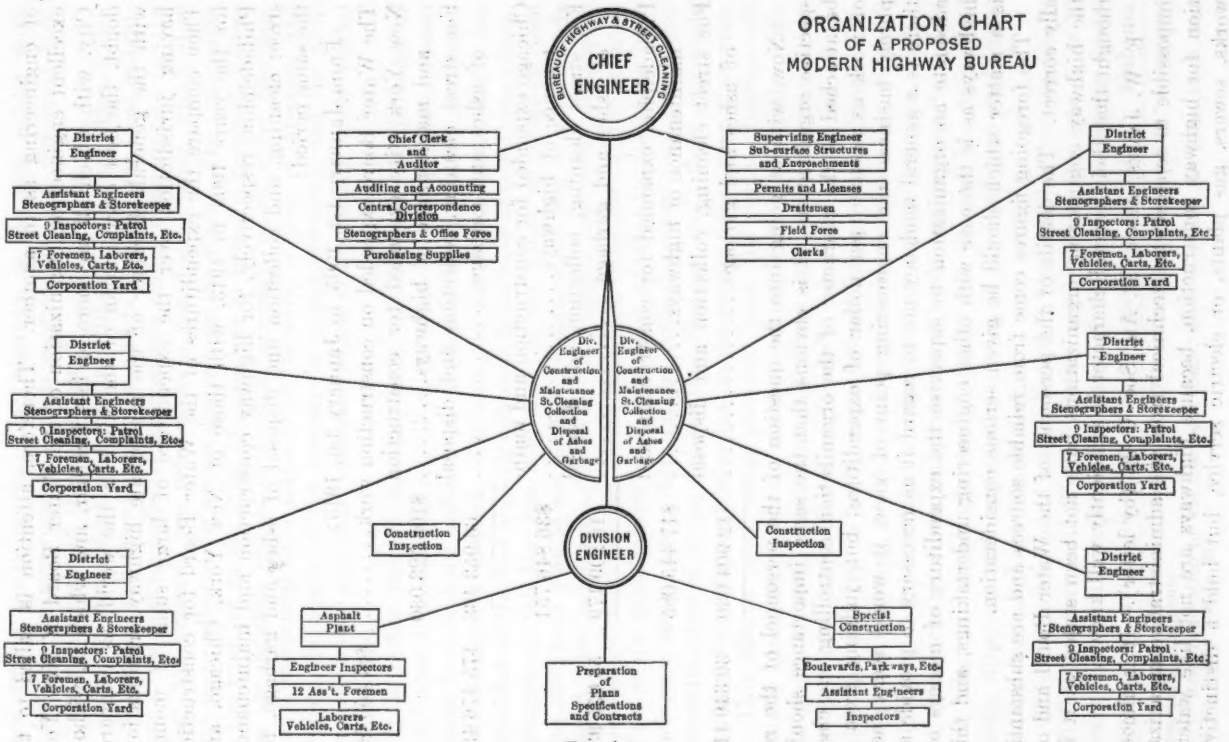

FIG. 1.

Mr. Connell. of engineering as in another. Therefore, attention is called to the excellent engineering organization of the Water Board of New York City, with which all are more or less familiar, and which is, without doubt, the best organization of the kind in the world, as compared with the general character of the inadequate highway organizations having jurisdiction over the expenditure of large sums of money. Then compare the expenditures of the Water Board for construction for the years 1905 to 1913, with those of New York, Chicago, and Philadelphia, respectively, for highway construction and maintenance, street cleaning, and collection and disposal of ashes and garbage, for the same period:

From January 1st, 1905, to January 1st, 1913:

The Water Board expended on construction work.....		$87 550 540
New York City expended for construction and maintenance of highways.....	$108 523 030	
For street cleaning, collection and disposal of ashes and garbage.....	66 953 463	175 476 493
Chicago expended for construction and maintenance of highways.....	$36 846 751	
For street cleaning, collection and disposal of ashes and garbage.....	16 209 179	53 055 930
Philadelphia expended for construction and maintenance of highways.....	$17 445 085	
For street cleaning, collection and disposal of ashes and garbage.....	12 904 030	30 349 115

Now, without going into the question of the personnel of the respective organizations, it is obvious that the same importance should be attached to the character of the organization controlling one class of work as another, per dollar of expenditure; but, judging from a purely business and engineering point of view, it would seem that there is a general tendency not to regard it as necessary to provide as adequate an organization to supervise the expenditure of moneys on highways as is the case with other engineering undertakings, and this is a matter which should be given serious consideration.

The foregoing figures come from reliable sources and are substantially correct. The details of the personnel of the Water Board and of the highway engineering organizations have not been stated as it is thought that their general character is sufficiently well known.

Mr. James. E. W. JAMES, ASSOC. M. AM. SOC. C. E. (by letter).—It is almost impossible to separate the technical from the administrative organization for highway construction, because highways are not like waterworks, sewers, gas plants, or electric service, but hold a distinctive

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place among public works. The highway is the first public work undertaken by government, and its construction and upkeep is both a duty and a right of the community.*

In many States the road taxes are still paid in labor. This, which of course is a poll tax—is always a road tax. Wherever local enlightenment has become sufficient to provide a money commutation for the labor, the proceeds are returnable to the district from which they have been collected for use exclusively on the roads. The origination of the road tax in the form of labor was in part the result of following the English system of local government, which alone was generally understood by a majority of the early settlers in America, and in part the result of the poverty in money resources and the richness in brawn and time resources which characterized newly settled communities.

It is rather strange that the labor tax has persisted in some of the States, but wherever it still exists, there is usually a local reason which is considered sufficient to explain adherence to so antiquated and inefficient a method. That the old law stands is no doubt due somewhat to the fact that though conditions have changed, the administration of the law has continued in the hands of unskilled men who have been able to see neither its defects nor any way to remedy them.† Its persistence, however, has caused one very pronounced condition of mind among the citizens of a large part of the country. Most men's grandfathers who lived west of the Appalachians built their own roads; later, their fathers built their own roads; and, consequently, they of to-day believe that they can build their own roads, and they leave the road labor tax on the statute books year after year.

This idea is much more prevalent than the casual observer will usually admit, but when we see that communities, in which the traffic has grown to such proportions and weight as to demand a much better type of construction than formerly, continue to depend on the labor tax and try to build roads without proper engineering, without proper financing, without even proper tools in some cases, we catch a glimpse of the inertia which characterizes a persistent idea. When the rural county citizen undertakes road work, he continues to believe that what his grandfather could do, he can do, and he, more than likely, dispenses with engineering advice, if the local good roads association does not force him to seek it.

This attitude of mind was embodied in the road laws of practically all the Southern and Western States until within the past 5 or 10 years, and is still reflected in many of them. The following States continue to collect all or part of the road tax in labor: Alabama,

*In elaboration of this idea, Dr. D. F. Wilcox, in "The American City," makes some interesting comments.

†"Road Legislation for the American State," by J. W. Jenks, Am. Econ. Assoc., Vol. IV, No. 3, May, 1889, p. 23.

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Tennessee, North Carolina, South Carolina, Georgia, Florida, Mississippi, Texas, Louisiana, North Dakota, New Mexico, Oklahoma, Arkansas, Montana, Kentucky, South Dakota, Colorado, Idaho, Illinois, Indiana, Minnesota, Nevada, Missouri, Pennsylvania, and Wisconsin.

Almost entire lack of engineering organization in connection with highway building in a large part of the United States has been the direct result of these conditions. Further, whatever efforts have been successful during the past few years in putting new statutes on the State books, none has succeeded in eliminating entirely the common idea that the farmer is as good a road builder as modern days can furnish.

The county and town system of road administration which obtains almost everywhere, places the construction and maintenance of roads in the hands of the county commissioners. In many cases, the commissioners have authority to divide the county into districts and appoint a supervisor over each district. Frequently, a penalty is prescribed for a man who, if appointed, refuses to act as supervisor. These men are furnished with a list of taxable polls in their several districts, and their duties include the collection of the road tax. They summon the road hands under the labor tax system, on the conditions fixed by law, and see that the legal time, or so much of it as may be required, is spent in working the roads. Those road hands who choose are usually permitted to pay out at a rate varying from 50 cents to $3 a day.* The supervisor collects the commutations, gives a receipt, and spends the proceeds on the roads in his district. His accounts usually stand like this:

Collected in road taxes	$787.50
Paid out for road work	787.50

Generally, a large part of the sum charged to road work goes, by tacit consent, as his perquisite, into the supervisor's pocket.

There has been a somewhat strong movement away from such conditions during the last 10 years, and we find many new laws drawn to relieve us of such ridiculous and wasteful methods of road administration. Still, the remedy has not been general, by any means, and there are some interesting clauses in the new laws. In the Alabama Road Law of April, 1911, for instance, the State Highway Engineer "may * * * be consulted by the county commissioners," and in cases where projects involving State-aid funds cost more than $3 000, "the highway engineer, with the consent and advice of the proper authorities in the county, may prepare plans and specifications" for executing the work by contract.

In the Arizona law of June, 1912, it is declared that "all roads and bridges, when constructed, shall thereafter be maintained and im-

*The common rate is $1 per day, but it ranges from 50 cents in South Carolina, to $3 in Nevada.

proved when necessary, * * * under the joint auspices and direction of the State Engineer and Board of Supervisors, * * *." Mr. James.

The Virginia laws of 1904, 1906, 1908, and 1910, are a respectable effort to do better things, but, from the limitations placed on the constituted road officials, it would seem that the General Assembly believed that the county commissioners knew more about road engineering than the State engineers whom they might employ under the law. Iowa has recently passed a new road law (April, 1913) which is a departure in legislation. It is the most systematic effort to correlate road work done at county expense that has yet been made. It could well be studied by the legislatures of three-fourths of the States. The new State of New Mexico goes one step farther in its road law of 1912. A State commission is created, and one of its duties is to appoint all county road commissions, each of which is composed of three members.

In general, however, except where State funds are involved, there is no engineering supervision required over county road work in the rural sections of most of our States. The counties thus conditioned spend, according to the most accurate reports attainable, about $61 000 000 annually on their roads. This is 40% of the annual road revenues of all counties.*

As soon as a State organizes a highway department and sets aside funds to aid the counties, the first systematic engineering organization is developed. The types of organization are simple and rather uniform for most States. They are confined to two common systems: one has a corps of assistant engineers reporting directly to the chief engineer; the other has a group of division engineers reporting directly to the chief. In a few cases, notably in Ohio and New York, there are deputies having general charge of a department of the work, as construction, bridges, maintenance. In these cases division engineers report to their respective deputies.

Under the division engineers there are resident engineers and inspectors assigned to projects.

A review of these actual conditions presents several questions well worth discussing. It is obvious that much of the reluctance on the part of poor counties to seek engineering advice is because there is no man available in the county, and an outsider—besides being an outsider, which frequently is considered more or less objectionable in itself—cannot be secured at a compensation warranted by the county revenues. As we see, therefore, it is not until the State steps in and gives the county funds that there is any engineering supervision of a large part of the county work. Cannot some simple engineering organization be created to deal with this situation?

Cf. Jenks, *supra*, p. 43. The principal exceptions are Massachusetts, New York, New Mexico, and Iowa.

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At a hearing of a legislative committee in a Southern State, the writer had an opportunity, about 2 years ago, to suggest a plan that he still believes has merit. The plan comprises a highway commissioner, paid by the State, drawn, if possible, from the staff of the State university or mechanical college, so that the additional salary would be small; a corps of division engineers assigned to groups made up of from eight to ten counties.

The commissioner would have general supervision of the division engineers, and they would have charge of all road work in the counties of their divisions. Many of these counties never would undertake macadam construction. Their roads, because of lack of local funds, must remain earth, sand-clay, or gravel for many years to come. The duties of the division engineers would be to supervise the expenditure of the county funds. The commonest needs in most rural counties are: the substitution of some material besides short-leaf pine for box culverts, the construction of adequate ditches, the proper crowning of the traveled way, direct and regular alignment, the intelligent mixing of sand and clay, the drainage of spring holes and swampy places, and the removal of vegetation from old mudholes. The division engineer's duty would be to see that suitable construction gangs are organized to work over the county and do their work properly. Labor taxes should be commuted by cash payments, and the money used to pay the road gangs. The old supervisors should be discontinued, and a group of foremen substituted, who, if possible, should be retained permanently, and trained in their work. When bridges or culverts are needed, the division engineer or commissioner can furnish plans, and the division man should see that the work is done in a correct and efficient way.*

This plan would much reduce the cost of engineering to the counties by a general sharing of expenses, and would be productive of good results in several directions. It would do away with the supervisor system, and would tend to remove the work from political influences. It would concentrate expenditures, and prevent the general custom under the ordinary county method of working every road in sight to please as many voters as possible, with the obvious result that the work is spread out so thin that the rainy or winter season destroys it all. It would create some standardization and begin the elimination of hopelessly ineffective methods used year in and year out by the county officials without apparent betterment of the roads. It would create in time some sentiment favorable to the employment of private engineers and a general raising of the standard of all public works. This latter circumstance would be a most happy result, for the need of adequate engineering in small municipalities through-

* Jenks, *supra*, p. 48.

out the South and West, and indeed almost everywhere in the country, is glaringly apparent on the most casual investigation. Mr. James.

When State funds are appropriated, conditions are at once produced that direct attention to another question. Shall the organization distinguish between construction and maintenance? The last report made by the New York State Commission recommended increasing the number of divisions and combining the responsibility for maintenance and construction. This recommendation was interesting and significant, coming from an organization which has had such a varied history as the New York State Commission. The State Legislature, acting in part on this recommendation, passed a law permitting an increase from six to not more than nine divisions, and otherwise embodying the recommendations. The type of commission was also changed, and a single responsible head provided. The present highway department in New York works under this new statute.

The labor organization and type of inspection required on construction, on one hand, and on maintenance, on the other, are so widely diverse that it appears to the writer that some separation of the two classes of work can be advantageously made. In the first place, if the division or district heads do not have to split their duties between the two classes of work, each can cover a larger territory. Further, if there is a system of standards used, the maintenance divisions can be enlarged beyond the size of those possible for construction divisions. This is so because, in any system so large as to demand division or district organization, the quantity of work makes construction by contract practically compulsory, and contract work requires constant and detailed inspection. On the other hand, maintenance by either the patrol or gang systems is force account or administration work, and as the patrolmen, foremen, and gangs are trained, they require less and less minute supervision. As a further consideration, it is not a bad plan by any means to establish as soon as possible the separate identity of the maintenance organization; because at last this will be the only one left, as in the case of the French *Routes Nationales* today. Trained engineers are generally required to take residences, whereas a division, when entirely under maintenance organization, would probably require no such resident engineers. In very large maintenance divisions assistants might be necessary to handle the larger re-surfacing projects, but in most cases trained foremen would be able to do a large part of the work under general orders.

On the other hand, an indisputable advantage comes from the vertical organization of divisions. If an engineer has entire charge of a restricted mileage during construction, he is doubtless the best prepared man obtainable to handle the maintenance of the same sections. He has watched the construction and knows the weak spots, the bad soil, the softer stone, and the doubtful drainage arrangements.

Mr. James. He knows the county thoroughly, and should have developed amiable relations with local officials and the citizens along the roads. All these matters would count much in a man's equipment for division work, but there is no insurmountable reason for a division man not leaving record maps sufficiently complete to inform a successor, instructed to take over roads for maintenance, in all the major and many of the minor conditions along the sections under observation. This matter of small-scale record maps is capable of large and valuable development. Further, a man detailed to maintenance, and concentrating his attention on it, should soon acquaint himself with the conditions in his division. It is seldom that labor used in construction stays to maintain, and, therefore, no especial advantages in the line of securing skilled laborers can be expected under either system. A new labor and supervisory organization will generally have to be created, and, in most instances, be recruited from the local supply.

Another matter that, from the standpoint of efficiency and economy, is of especial importance, is the place occupied under the State organization by the county road administration. Three States, New York, Iowa, and New Mexico, have statutes specifically placing the county road engineers or supervisors in the State system. There is no denying that this innovation (for, in the present condition of our State road laws, it can certainly be considered such) has little about it to be unfavorably criticized. Of course, when within a State the question of local autonomy comes up in any form, we always find some who oppose any restriction on home rule; but, if we analyze the basis of home rule, we find it chiefly concerned with matters of conduct. Now, road building is not a matter of conduct. Our fellow-citizen does not understand why his road money should not be economically expended by competent engineers. Now that two or three States have led the way, it may be hopefully expected that there will be a further move in the direction of placing some check on the indiscriminate expenditure of county funds under a local system which is worse than simply inefficient. This can be done, even when no State-aid feature enters in as a sop to the county. When such allotment does appear, there is no reason why supervision should not extend beyond the limits of the State-aid roads.

In Iowa the county engineers and the county officials, as far as road matters are concerned, are under the supervision of the State commission. In New Mexico the county road commissions are appointed by the State commissioner. In New York, county and town supervisors are under the division engineers, and in Illinois, the county engineers are appointed from a list of eligibles satisfactory to the State commission.

Just how far the State organization shall be indicated by law is a matter of debate. The writer believes that few States err on this point

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at the present time. Generally, except in three or four cases, the detail of the highway department organization is not carried beyond the State highway engineer under various titles. The details of organization are left to the administrative heads and the engineer. This is as it should be, until we find some organization that has details of particular merit capable of wide application. This freedom, however, does not preclude placing the county road authorities by law under the supervision of the State organization.

Many of the State departments to-day are advisory or nominally supervisory in their duties, and it is this restriction which, in spite of freedom in departmental organization, restrains their usefulness in so many cases to almost negligible limits. Where the great gain in efficient expenditure must come is in the county work, and the State engineer is helpless, however competent he may be, unless he is given some element of authority over the local activities. Merely advisory work is very often gratuitous. Some county officials consider it almost meddlesome. It is these local Solons that good organization must circumvent.

A review of the several State laws shows that the highway departments suffer from one very general condition. There are few single responsible heads. We are familiar enough with actual working conditions in some States to know that the work suffers, or has suffered, seriously because responsibility is not concentrated in one administrative official. It is very common to have the law stipulate that the engineer shall work in conjunction with an administrative official, sometimes even with an administrative body, of equal or greater authority. Arizona presents a case of this sort. In some States work is being done successfully, because, with a poorly devised law, a commission of three or five members has turned the work over to one of their number, or to a chief engineer, who acts practically as a single head. This has been conspicuously true in Virginia. The ill success of a many-headed commission can be seen in Rhode Island. There a commission of five members, one from each county, has had active charge of work, and instances have occurred where commissioners, visiting construction in progress, have changed the orders of the engineer in charge.

With regard to details of organization under the commission or responsible heads, the laws in most cases leave the highway department free to adopt such system as seems best fitted to accomplish the ends and purposes of the statutes. Were there a general system of single responsible heads, many States under their present laws, could develop thoroughly efficient organizations. The responsibility for the organization would lie with the head, and he would generally be unrestricted. There are thirty-eight States which have some form of highway department, but it is doubtful if more than fifteen of them

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ean develop efficient highway organizations, because of the divided authority and responsibility residing in several heads.

Mr. Bennett.

CHARLES J. BENNETT, M. AM. SOC. C. E.—With one or two exceptions, the speaker is impressed with the general soundness of the arguments presented by Mr. Connell. In the first place the formation of a standard organization for a highway department in a city is not entirely feasible. There are so many difficulties in the way of standardizing any particular department of a city that it would be impractical to consider any more than general principles to be used in all cities, instead of trying to standardize a system for one department in all.

For instance, a standard system applicable to New York City, Chicago, or Boston would certainly not apply to a smaller place, on account, not only of the size of the city itself, but the general geography, characteristics of the people, and the system of government. It seems plain, therefore, that if a standard system is to be devised, all cities in the United States should be classified and a system of organization for all departments formed, so that the entire operation of city government could be along standard lines for different types of cities.

This seems to be an ideal which cannot be reached, and the problem, therefore, must be to apply general principles to engineering organizations which shall tend to improve conditions which are bad at present. These applications should also furnish engineers with information which can be classified by engineering societies, so that costs and systems may be contrasted and certain valuable improvements in results gained.

There is no question of the need of an engineer to supervise not only highway departments, but all public works of any character. Therefore, a Municipal Highway Engineering Bureau should be a part, and a very large part, of an Engineering Bureau of Public Works, and public works of all kinds should be under the supervision of a trained engineer.

This principle, if applied to cities, will work with incalculable benefit, both in the improvement of the construction of public works and in their economical operation after completion. Granting this, therefore, Mr. Connell's statement as to the need of an engineer to supervise all highway work is evident, and the supervision of this work, as he states, should include, not only the actual construction and operation of these public works, but also the design of the different structures as well as the outline of the general plan to be carried out in the expansion of the city.

Now, a word as to the qualifications of an engineer in public service, and the difficulties of securing one who is competent to follow up work of this class: Engineers by training should be fitted to do all things necessary for the operation of such a magnificent department

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as has been conceived in control of the public work of a city or State. That means that the engineer must be not only a technical man but a business man who can see both the scientific and financial sides of the problem, and determine the advisability of spending any given sum of money for a certain purpose.

The second qualification means that the engineer must take a broader view of public work than he generally does.

In considering the operation of any city or State engineering department (and the ideas here outlined apply, it seems, particularly to the highway department), there is need of a better training for engineers in keeping accounts and supervising routine office work. The success of any highway bureau, or any other bureau which is under the supervision of an engineer, depends on its accounting system as much as on any part of the whole situation. The expenditures must be kept within appropriations, and work must be planned and executed in such a manner as to gain the full benefit, so far as possible, from the expenditure of moneys allowed. Therefore, the engineer should be trained, and highly trained, along business as well as technical lines. The natural tendency would be for an engineer to be too much given to minutiæ and complicated methods. The endeavor should be to devise a system of accounts as simple and free from unnecessary complications as possible.

This discussion has followed general lines more than the particular ones specified in the topic, but these ideas—which must be considered as general, rather than specific—can be applied to a highway engineering organization.

In connection with the discussion on highway work, both municipal and State, it is interesting to note the diversity of opinion among engineers as to the value of the work done. Within the past year a prominent engineer has told the writer that there are very few engineering problems to be faced in highway work; that this work is of the simplest nature, and, by inference, almost any one could become a highway engineer.

To the ordinary man's mind, the highest type of engineering would be that which would do, in the best manner, both the simple and the complicated things.

There are comparatively few very large structures or projects to be carried out by engineers, but, as indicated by Mr. Connell, the largest amount of money spent in engineering work is for the operation of municipal and State highway departments, and the results of this work are open to critical observation by all who are in possession of five senses.

Although the engineering problems may be simple, the problem of spending public money in an economical manner, is, by far, the hardest one which the engineer has to solve, and if this can be solved

Mr. Bennett. by improved business methods, as well as improved technical methods, it seems that we will have done some of the highest class of engineering work.

In conclusion, the speaker endorses Mr. Connell's statement, that the highway or other departments of cities or States should be freed from political influence, and that the spoils system should be entirely eliminated in the operation of these departments. Only a moderate amount of success can be attained when any system of politics, whether it be good or bad, is allowed to interfere with a municipal or State department, and when these departments are entirely free from political interference, then, and then only, will ultimate success be gained.

Mr. Dean. A. W. Dean, M. Am. Soc. C. E.—Very recently the word "efficiency" has become a particular favorite, not only with those who are directly concerned with State and municipal affairs, but with all who are interested in the administration and execution of public works. Witness the large number of governing bodies which have created commissions or boards of efficiency and economy to analyze and criticize the acts of public officials, and, by their advice, minimize waste and extravagance in the expenditure of public funds.

No department in State or municipality lends a greater field of action to such a board than the highway department, when competency is not the prime requisite in the selection of heads and assistants; and, on the other hand, no highway department consisting of competent men selected for their particular fitness for the positions they occupy will furnish a fair-minded efficiency board with any cause for reprimand or public criticism.

The preceding statements are made only to emphasize the fact that no highway organization can be successful unless competency is the first and only consideration in the selection, not only of the head, but of the entire body of the organization. Competency and efficiency are inseparable terms, for without one the other cannot exist.

A detailed description, either in words or graphically, of an organization adaptive to any and all conditions is obviously impossible. Highway laws in the several States vary widely, as do also conditions to be met in municipalities of varying wealth and population.

Supported by laws prescribing a well-defined policy in the expenditure of State funds, a single responsible head is preferable to a body of three or more; and, on the other hand, if the State has not, by legislation, fixed a general policy, a body of three may rightly be considered preferable. With or without a prescribed policy, a single responsible head with an advisory council having limited powers should prove efficient in carrying out large or small undertakings.

In a municipality it has been well demonstrated that a single responsible head is sufficient.

No organization can be long successful unless it is arranged below the head in military order. If the organization is small, the head may act as the chief executive; if large, there must be one chief executive charged with carrying out the general policies and orders of the head, as a division of responsibility at this point tends toward duplication, uncertainty, and consequent inefficiency. Mr. Dean.

Divisions and sub-divisions below the chief executive officer are dependent on the scope of the work to be done; in State highway organizations a suitable number of division engineers are charged with responsibility for both construction and maintenance in their several divisions. In municipal organizations each division should include the entire municipality, with a division engineer in charge of each branch of the work, that is, one in charge of bridges, one for street construction, one for maintenance, etc. For large undertakings the division may be subdivided into districts, with district engineers in charge responsible to the division engineers.

To summarize and further present the matter briefly, the following outline of highway organization is submitted:

1.—A single commissioner, or three commissioners, as quite definitely recommended above;
2.—A single responsible chief executive officer;
3.—As many divisions as may be necessary, each under a division engineer;
4.—Sub-divisions or districts, each in charge of a district engineer;
5.—Minor assistants.

All appointments should be governed entirely by fitness; there should be no changes in the personnel, except for proper cause; there must be no division of responsibility; all orders must be transmitted as in military practice.

HENRY W. DURHAM, M. AM. SOC. C. E.—It is of interest to have confirmed by Mr. Connell's search, the fact, apparent to all who have been charged with city street maintenance and repairs, that there exists little constructive criticism based on broad lines as to the methods of carrying on such work. Destructive criticism abounds. Mr. Durham.

Reports of citizens' committees denouncing indiscriminately pavements of bygone days, or the non-use of some special, possibly patented, cure-all street surface, the promoter of which has had an accelerator at work, illustrated by views of the depression where the prominent citizen's automobile was bumped, and accompanied by half-baked legal recommendations based on the American citizen's ineradicable impression that the millennium can be attained by "passing a law"—these, and the annual reports telling what has been done with the tools we have at hand, and the textbooks that tell much about good

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pavements, but little as to paving organizations, comprise most of our highway literature.

There have been, it is true, studies made and plans advocated by efficiency experts for the better handling of the routine work of existing highway departments, or for their improvement in details, but the results of Mr. Connell's investigations would seem to indicate that little has been written on the fundamentals of organization and the proper scope required of the department in charge of highways in a great city. This is possibly due to the fact that in America the demand for improved street surfaces, caused by the increase in number, weight, and speed of vehicles using the highways, is coincident with the arrival of conditions causing their upheaval to a maximum degree. In no other part of the world at the present time, and in no previous period of the history of the United States, has there been anything paralleling the amount of rebuilding and shifting of what were supposed to be established trade and social centers, that is to-day going on in all large municipalities.

While the general public is demanding the construction of streets with absolutely smooth surfaces for availability for motor race-tracks, that shall be unyielding and without surface wear, so as to endure for all time, constructed over subways and pipe galleries to avoid all future surface disturbances, it is at the same time abandoning whole districts devoted to residences and light traffic, and filling them with tall office buildings which, when the expected office occupants fail to materialize, are turned into factories having a population in some sections of New York larger than the great mill towns of New England; and is demanding that these buildings have adequate service for telephone, electric light and power, and water supply, including high pressure for fire protection; and, finally is planning new lines of subways to carry away the surplus population brought to these districts, and to open up new possibilities of change. To harmonize these conflicting demands would appear to be an impossibility. It is under such conditions that officials charged with the maintenance of municipal highways are to-day carrying on their tasks.

To construct and maintain pavements under present-day traffic is an occupation in itself. When to that is added the fact that in no great city can the highway officials determine with certainty whether the pavement laid to-day will be suitable for the traffic to be found using it 5 years hence, it will be seen that they are entitled to the benefit of the best constructive criticism available, and that the fault-finder, who devotes what knowledge he may possess to enlarging his own newspaper reputation by the cheap method of criticizing obvious defects, is actually blocking progress.

Mr. Connell has taken up the problem in the right way, and has indicated an outline for the commencement of efficient highway or-

ganization. It may be of interest to have the subject also treated from other standpoints. In the course of an investigation of municipal highway work in European cities, made during the past year, some points were noted as to their methods of solving this problem. Mr. Durham.

Two general methods are followed in different parts of Europe:

1.—A strongly centralized authority under which the entire city is sub-divided into a number of smaller unit organizations, each within its district having charge of all municipal work, closely governed by the central office;

2.—Local control where, in the city as a whole or as sub-divided in each of a number of independent units, a single head has control of all city work, with sub-divisions, each separated and independently charged with a single class of work.

The first system is best exemplified in Paris. In the organization of the street service and for all construction work in the highways, it is divided into seven sections, each headed by a resident engineer who has control, not only of paving work but of the laying of water pipes, sewers, and the supervision of any sub-surface work of private corporations. At the head of the bureau of street service is an Inspector General with necessary deputies and office organization, having under him several chief engineers, one in charge of public streets and street lights to whom the resident engineers primarily report; another at the head of the work of street cleaning; one in charge of gas and electric light mains; the fourth in charge of water and sewers.

Each chief engineer has the necessary office force, and several resident engineers are under his orders for special work and for the management and maintenance of work outside the city; but, within the city, all construction work on the streets is under the direction of the street service engineer in charge of each district. Each in his own section takes care of the paving work; lays water pipes; builds sewers; and supervises duly authorized construction on the part of gas and electric companies, so that no pavement can be torn up except by his orders or with his permission.

In London, on the other hand, the control is almost entirely decentralized. There each of the thirty different boroughs has its own city engineer or surveyor responsible only to the Borough Council for orders and directly in charge of all street paving, repairs, cleaning, and lighting. The works of general utility to the entire city, such as water supply, main-drainage sewers, and municipal tramways, are controlled by special boards independently of the boroughs, and do any necessary street work under the supervision of the local borough authorities. Smaller English cities have usually a city engineer, charged with control over all the city works, having under him a Department of Highways headed by a road surveyor, and an organization

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of assistants whose duties are confined to the maintenance, repair, and cleaning of highways. With some modifications, the system in the German cities is very similar.

In the first of the two systems there is one responsible head over all classes of work carried on in each unit—the unit being some convenient sub-division of the entire city—and though the different divisions of city work are each under special organizations, the actual execution, in theory at least, is controlled so as to render lack of co-operation or interference of different classes of work impossible.

In the second system, in the city as a whole, or in each of its independent units, in the case of large cities, there is centralized control to the extent that all construction, with the exception of water-works, is under the direction of a city engineer, but the special varieties of street work are usually carried on by independent sub-units. Of course, many exceptions are found in any necessarily brief and general classification like this. For thoroughness and theoretically perfect, systematic, and orderly procedure, the French method of organization seems to leave nothing to be desired.

The city work of London, on the contrary, is carried on under what is probably the most illogical and unsystematic arrangement that could be devised, but, judged by practical results, London presents probably the best appearance of any city in the world, as far as the streets are concerned, and Paris is constantly in a torn up and poorly maintained condition.

In Berlin, a city about equal in size to Paris, but divided into a number of independent city or borough governments, without a central control, but very thoroughly organized as units, the results show good conditions in some districts and poor in others. The practical deduction to be drawn from the comparison of these methods, not on paper but as worked out in the three great European cities, is that no system will produce results automatically, and that good results can be produced under adverse methods by a people accustomed to orderly procedure.

It will be seen that the scope of a department in charge of public highways can be very broad or very much restricted and still be in accordance with good practice in some prominent city.

Mr. Connell's plan seems to follow the French system, but without going far enough for completeness. If the work of the street department includes cleaning, it may well go farther and take in sewer, lighting, water and other city works requiring the use of the streets. If we believe, like Sterne, that "They order this matter better in France," we may well consider that a model city works department should be based on that of Paris, which, after co-ordinating all possible interests, furnishes ample opportunity for the card-index and loose-leaf expert to devise interlocking systems to his heart's content

until he arrives at his Utopia, where the Inspector Generalissimo, by a system of push-buttons at his desk, can run his department and never see the work. Mr. Durham.

The weak points which make this system difficult of application lie in the question of personnel. We have in the United States no trained class of public works officials, no body of Government engineers organized like the French Corps of Bridges and Highways, and we have frequent elections and still believe at heart that "to the victor belong the spoils" including the right of spoiling the work done for us by the vanquished. It is hard enough to find one civil service man to fill a responsible position; to find seven equal and uniform would be almost impossible. In the light of possible conditions, it would seem to be a better policy to concentrate along all classes of work, organizing the Bureau of Highways into sections for doing specific things in charge of the men best qualified to do them. An outline of such an organization, based on experience in Manhattan Borough, but making improvements in some existing conditions, is somewhat as follows:

The head of the Bureau of Highways has charge of all construction and maintenance work on street surfaces between building lines, and issues and controls permits for necessary street openings of whatever nature, under the direction of a Commissioner of Public Works whose authority includes, besides highway construction, the sewers and other sub-divisions and in whose office all financial matters, including auditing, contracts, etc., are taken care of.

To avoid duplication of work, a single Bureau of Design and Survey carries on all necessary work for the different construction bureaus in the department, its work for each, however, being under the direction of the head of that bureau. This applies, not only to the making of preliminary and contract plans, but to the furnishing of parties for surveys, giving grades, and making measurements for final estimates.

Under a Chief Clerk is the control of all correspondence and records of the bureau, including the necessary organization for filing, recording, and handling data for the maintenance of a live record as to street conditions, necessary repairs, etc.

As has been pointed out, the control of street openings is an essential preliminary to the work of any highway department. No system, however complete, will be of more value than as a historical record unless some plan lies back of the causes necessitating such openings. It will be essential ultimately that our great cities realize the desirability of the adoption of a systematic plan for sub-surface structures for new districts and take up the problem of an orderly re-arrangement of these structures in the most congested sections. Thus far, charter rights of individual companies and the unwilling-

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ness of various departments to co-operate with one another have rendered futile any attempts in this direction, resulting in a tangled mass of sub-surface structures in our central districts, with locations frequently unrecorded, and the repair or extension of which requires an undue amount of street surface disturbance. Pipe galleries under all streets are a dream of the idealist, but an orderly planning which shall place under footways the lighter house-service lines, leaving the large mains under the roadway at regulated intervals, and shall do away with unnecessary duplication of facilities in any one street, and a limited provision for pipe galleries in the most crowded sections, is not impossible of attainment. It will require, undoubtedly, amendments to the charter, both of municipalities and corporations, and the vesting of the right to order changes in the body controlling the city policy. With such a condition existing, the permit office of the Bureau of Highways will have actual and not merely (as at present) nominal control over all street openings. This can be done without interfering with the rights of any other department or corporation to get at their structures for emergency repairs. The usual procedure will involve application to the permit office in advance of making any opening in the pavement, and a deposit of sufficient money to cover the cost of restoration, inspection of sub-surface work, back-filling, and subsequent repair and maintenance.

After street openings are properly controlled, the making of repairs necessitated by ordinary wear is of next importance. For convenience, this work has been placed in two divisions: one having charge of all repairs to block pavement, including stone block, wood and asphalt blocks, a large part of which work can be done by gangs with a limited quantity of new material, and repairs to sheet-asphalt pavements, which require the use of an asphalt plant, either owned by the city or operated by contractors. To lay out the work of these divisions properly, a system of street inspection is essential, and this is done by a division of patrol inspectors, each having a definite district and making daily reports of all defects, which are immediately plotted on an office map from which the day's work for the gangs can be laid out to the best advantage. Another division looks after all matters pertaining to sidewalks, whether carried on by the city or the property owners, and has control of such other matters as street signs, removal of incumbrances, etc.

Finally, as the perfect non-wearing pavement has not, as yet, been discovered, it becomes necessary to provide for renewals of street surfaces from time to time. This, for the present at least, is best done by contracts, under the supervision of a division engineer with a necessary organization of assistant engineers and inspectors.

Whether the street work is performed by city forces or by contractors, the purchase of new materials and their conformity to specifi-

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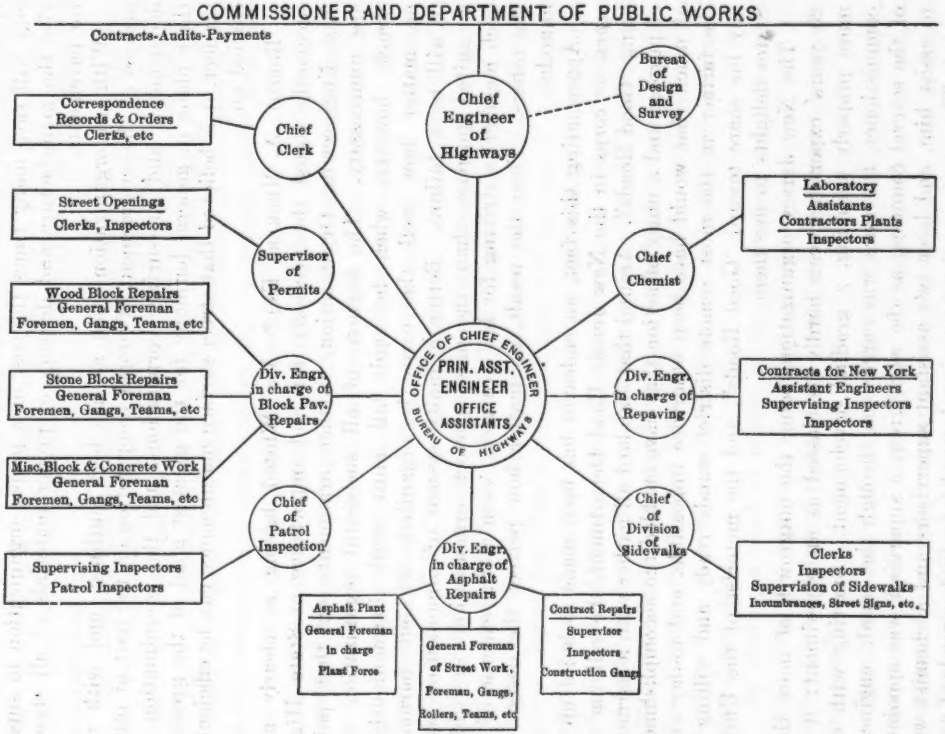

FIG. 2.

Mr. Durham. cation requirements is essential. Hence, one of the most necessary divisions is the laboratory and plant inspection force in charge of the chief chemist. Losses due to mistakes or inadequate inspection in this division may amount to many times in value what would occur from omissions or carelessness of inspection on construction, and the city will save many times the cost of a proper organization to supervise the manufacture and delivery of proper materials of all classes for pavements.

With an organization such as has been outlined, and with the proviso of the enforcement of proper co-operation on the part of other departments and sub-surface corporations, and the co-ordination of their plans in general harmony for the greatest good of the greatest number, it is believed that street surface maintenance can be efficiently carried on.

Mr. Meeker. ROBERT A. MEEKER, ESQ.*—Mr. Whited having so clearly and succinctly stated the necessity for and importance of State Highway Engineering Organization, further arguments on that point are unnecessary. The keynote of all successful organization was struck, however, when he said "all must work harmoniously"; no matter how well the corps is organized, without harmony it will be a failure. Better a faulty system of organization with a united purpose than the most perfect organization in which each member is striving for his own glory, even at the expense of all the others. Then the result, no matter how perfect the design, is anarchy.

Appreciating this fact, an endeavor has been made to build up an *esprit de corps* in the New Jersey Road Department, the rallying cry being "Good Roads". Around that standard all gather with a harmony of purpose and a unity of action that have enabled the accomplishment of much that would have been otherwise impossible, and to-day even the farmer in the most remote district stands ready and willing to pay his share toward "Good Roads", and the motorist of the United States delights to use them.

The New Jersey organization, being the outgrowth of more than 20 years' experience, can hardly be classed as an experiment; it is, more properly speaking, a gradual development. Starting with one commissioner, the work was carried on through the county engineers of the several counties; as the work grew, a supervisor was appointed to assist him, and later two assistant supervisors and a chemist were added. Then, owing to the increasing demand for a higher grade of road, caused by the advent of the automobile, a larger organization was demanded, and the re-organization under the present Commissioner was the result. It consists of the Commissioner, who is chief executive, the

* State Highway Engineer of New Jersey.

Highway Engineer, who must pass on the design of all engineering work, four Division Engineers, three of whom have each charge of a prescribed district and the fourth has charge of the bridges, a Chemist, who tests all road materials, twenty-one county engineers, who, though employees of the county, are also resident State engineers in their several counties, ten inspectors, and six foremen; the latter are to form the nucleus of a force that will be needed when the State highway system becomes an accomplished fact. Mr. Meeker.

The foremen and inspectors report in writing weekly to the Division Engineers, who in turn report to the Highway Engineer and he reports to the Commissioner, thus each has his alloted task and each his share of responsibility. The county engineers might properly be classed as brigade commanders having command of the work in their several counties and being subject only to general orders from headquarters as to standards that must be met.

The State highway system, just referred to, is also a natural outgrowth of the work, its aim being to unite the several county seats and to extend the main highways to the borders of the State. This being such a large proposition, it was deemed wise to establish a Highway Commission which should determine the general plan and location of the work. This Commission consists of the Governor, the President of the Senate, the Speaker of the House, the State Treasurer, and the Commissioner of Public Roads. This Commission has laid out a system of roads 1 500 miles in length, most of which are already improved, hence the State's principal task will be one of maintenance, which, owing to the fact that New Jersey lies between the two largest cities on the Atlantic seaboard, will always be a big one. Owing to the fact that many of our main highways have been laid out for from 100 to 200 years, their exact location is often hard to determine, hence it was necessary to add a right-of-way engineer to the force, and his services have proven so valuable that, in all probability, others will have to be added.

In closing, a word of tribute to the contractors should be added, for to them the engineers owe much; it is their practical skill that makes the designs and plans of the most perfect engineering organization possible.

PAUL D. SARGENT, M. AM. SOC. C. E.—The one argument that has been most effective in every State in bringing about the creation of a State highway department has been incompetency and inefficiency on the part of local officials in securing economical results in the expenditure of highway funds. Consequently, every State highway engineering organization, to prove its right to exist, must be developed along the lines of efficiency, competency, and honesty. Politics, in the ordinarily accepted sense of the term, can have no place in such an organization. Mr. Sargent.

Mr. Sargent.

It seems to the speaker that the best results will be obtained and there will be more general satisfaction when the work is in charge of a commission of three members, provided they are men of sufficient breadth of view to understand their problem in its largest sense.

The commission should formulate the general policy to be adopted by the department, unless this has already been covered by law, and should pass on all large questions of a business nature.

Unless the commission is in daily session, which is rarely the case, it will be necessary for it to select an executive officer, who will usually be the chief engineer.

Besides engineering ability, this man should possess a good degree of tact, for meeting and dealing with the public, especially local officials. This is particularly desirable where highway improvements are paid for jointly by localities and the State for this always means that the State directs the work, and, unless handled just right, local officials will feel that their authority is usurped. Other things being equal, tact in dealing with local situations should be a controlling factor in selecting subordinates all through the department.

The chief engineer should have absolute control of the appointment of all subordinates, and, of course, they should report to him. The size of the organization will depend entirely on local conditions, that is to say, the quantity of work to be done in a given time and the area over which the work is distributed.

The following organization seems to the speaker to be about as small as can be expected to handle efficiently any considerable quantity of work: A division of surveys and plans; a division of design and construction; and a division of maintenance.

The division of surveys and plans will make all surveys and secure all preliminary data necessary for a proper design of contemplated improvements. This division should furnish plans of the road as it now is to the division of design and construction, where, after examining the locus, with plans, profiles, and sections in hand, changes of alignment and grade and details of construction will be determined.

The plans will then go back to the division of surveys and plans for completion, including preliminary computations of quantities, on which an estimate of cost will be prepared and bids will be invited.

At the same time the division of design and construction will prepare specifications, advertisements for bids, contracts, etc.

After the award is made and work is in progress, the supervision and inspection will be done by the division of design and construction.

It can be readily seen that in a small organization one man might handle both of these divisions, as their work is intimately correlated.

The division of maintenance, as the name implies, will look after this work especially, and it is suggested here, entirely on the supposi-

tion that both the other divisions are fully occupied with their particular problems. Mr. Sargent.

This whole scheme is based on the supposition that there will be no division offices, as there are in several State organizations, but that all engineering work is done at department headquarters.

To complete the organization, there should be a division of tests and one of accounting. Tests should by all means be reported to the chief engineer through the division of design and construction.

When the chief engineer is the executive officer, the division of accounts should report to him, in order that he may be informed of the fiscal operations of the department.

When a State highway department is in charge of a single commissioner, and a competent civil engineer is selected for the position, the scheme of organization herein outlined will be applicable by substituting in this discussion, the word "commissioner," for "chief engineer."

W. W. CROSBY, M. AM. SOC. C. E.—In an efficient organization for producing physical results from men and materials, the responsibility for the results must be clear—if possible, individual—and the authority and resources in any case must be sufficient, not only to equal or balance the responsibility, but also, in order to be on the safe side and to avoid any arguments in the matter, to be appreciably in excess of the required duties and responsibilities. Mr. Crosby.

In highway work, a division of the consideration is frequently made between Construction and Maintenance, and, perhaps, naturally, up to comparatively recent, if not actually the present, time, the bulk of the investigation and discussion in the United States has seemed to center under "Construction". It is beginning to be realized, however, that "Maintenance" is worthy of at least equal consideration, and that the proper organization of the maintenance forces is of prime importance.

The speaker, after long experience and wide observation of the experience of others, has had at least one conclusion clearly crystalized therefrom. That is, that, not only for the sake of securing prompt satisfaction in the results of the expenditures for labor and materials toward maintenance, but also for the sake of economy and efficiency in the long run, it is necessary to separate the maintenance work from the construction, and thus clarify the responsibility for the proper performance of each.

Naturally, there is no need for a State Commission or Commissioner for the Construction and another for the Maintenance, nor of two Chief Engineers on these lines; the segregation may properly begin immediately below the Chief Engineer, or it may not begin until the Division Engineer, or even the Resident Engineer, is reached. Local

Mr. Crosby.

conditions and due regard for the rest of the whole organization of the department will decide this point. Ultimately, however, the separation of work and responsibilities must be reached.

The speaker has worked up through the matter from the beginning, of all construction work and no maintenance, to the point where the maintenance mileage far exceeded that under construction, and he has tried the various methods of placing the responsibility for the maintenance on the construction engineers, or maintenance engineers, and on others. He has had the maintenance work done by patrolmen, by gangs, by contract, etc.; but satisfaction has been approached only through such arrangements as made the maintenance work and responsibility for its proper performance the sole duty of the employees up to a certain point where dependence could be had on that individual's appreciation of its proper relation to the importance of construction, and on whom the construction demands were not so great as to incite any neglect of the maintenance.

As has been previously stated by the speaker:*

"The importance and economy of prompt, efficient and sufficient maintenance cannot be overestimated in road administration, and this is especially true in the case of modern roads under present traffic conditions. More of the existing defects in roads today, whether such defects be those of appearance, of comfort, or of economy, are due more to weak points in maintenance than to deficiencies in construction or to any other cause.

"The requirements of maintenance work demand the careful performance of little things and the prompt attendance to such; persistence and continuity of action; good judgment and craftsmanship in actual work; and a high regard for economy and the old fact that 'many a little make a mickle'. Maintenance work contains far less of the spectacular than does construction, and thus some other stimulus is frequently needed by the workers in it that interest may not flag. Without some such incentive, the results are almost sure to be unsatisfactory."

Another matter to which the speaker wishes to call attention, in connection with the discussion under Organization, is that of the duties of resident engineers and inspectors.

The differentiation between these two cogs of the machine does not seem to be always as clear as it should be, and frequently, from lack of such clearness, duties seem to be extravagantly imposed on one or to be loaded on the other so as to cause a failure in the operation.

A resident engineer is generally supposed to be a man of some experience and skill in the work under his charge, and who, on account of such expertness, is delegated from above certain (limited) authority for making decisions and for directing the details of his work so as to facilitate the progress of the latter and relieve his superiors from the

* Report to the State Roads Commission of Maryland, 1912, p. 101.

necessity of considering and deciding minor details when once general directions have been issued in one way or another by them. It should be noted that ability, experience, firmness, good judgment, and tact are primary requisites for a satisfactory resident engineer. Mr. Crosby.

An inspector, on the other hand, is merely the eye of his superior, to whom he reports, focussed on the particular work on which he is stationed for the purpose of observing or inspecting its performance. He is instructed, by means of copies of the clauses of the specifications that apply to that work under his observation, or by other means, as to what is required for satisfaction of the contract in such work, and, in case of any apparent evasion or violation of the provisions of his instructions, is or should be directed to call the attention, first, of the contractor and, secondly, of his nearest superior with authority to the matter.

An inspector, as such, has or should have no authority over the work delegated to him, and his responsibility is, or should be, accordingly limited to reporting promptly on the facts as he notes them. To delegate authority to him makes him no longer an inspector but rather a resident engineer of more or less limited field, as the case may be.

It is not the speaker's intention to provoke discussion of the etymology of the two expressions, or to quarrel with their various uses, but rather, by the foregoing, to illustrate differences of meaning that may be assigned to each for the main purpose of clarifying and, as far as possible, lending force to the following remarks.

The speaker (and doubtless those to whom these remarks are addressed) has continually heard references to difficulties supposed to be omnipresent between contractors and engineers, and much time on many occasions has been occupied in discussing such differences or sources of friction and means for their reduction or relief.

From a long experience, in capacities ranging upward from inspector to chief engineer, the speaker has become convinced that the responsibility for such friction rests largely in many cases with the chief engineer or other party responsible for the organization of the engineering forces for highway work.

In connection with the actual organization of the engineering forces, consideration of the specifications for the work itself must not be neglected, for, in part at least, they generally affect the operation of the machine or express in words what is graphically or mechanically shown by the form of the organization itself.

In a lecture at Columbia University a year ago, the speaker made the following statements which it seems to him are pertinent for repetition here:

"With a proper organization of an engineering department, including the definitely understood delegation of authority, the full expres-

Mr. Crosby.

sion of the work to be required by plans and specifications, the location of the responsibility on the proper party (the chief engineer) and this recognition of their obligations on the part of the contractors, ninety-nine per cent. of the present causes of friction between the latter and the engineers would be removed and in place thereof would be substituted the cordial assistance of the engineers to the contractors.

"To hitch up an experienced contractor's foreman, bent only on making money for his employer, on one side of the pole with a young, ambitious but green, inspector (perhaps the only one obtainable) on the other side, and then to turn this team loose and to expect them to keep steadily along the road toward success would be folly, yet it is often attempted, for valuable inspectors do not often remain as such as long as do foremen; but if the foreman realizes that the inspector is to call for help the moment he feels he is losing his footing and that the call will be answered by an experienced man fully capable of handling the situation, there is far less likely to be the need for such a summons. Few competent engineers and contractors cannot get along together peaceably and advantageously. The difficulties seem to come largely from mismatching, by improper delegation of authority, and from lack of clearness and definiteness in describing the work and materials to be furnished.

"The speaker knows from experience the difficulties of the young, enthusiastic, impatient resident in preserving a placid equanimity day after day when constant evidence of a desire on the part of a contractor to evade the specifications, to substitute 'something just as good' for the materials and work he has agreed to deliver, is offered. On the other hand, he sympathizes with the contractor who is constantly being nagged by an inspector over minor deficiencies, or when the inspector tries to 'run the job himself'. Perhaps it is human nature for most contractors to try out an inspector to see what shortcomings he will quietly submit to, and, having found out, to his advantage or loss as may be the case, thereafter to gauge his work accordingly. Perhaps it is natural for most contractors, after competitive bidding on public work, to attempt to substitute at every possible opportunity 'something just as good' for the article specified. But the speaker believes that much improvement in the relations between contractors and engineers would be had were the organization and specifications, as before referred to, made most explicit and then the contractors to regard their carrying out to the letter as much an obligation on themselves as the completion of the work."

Therefore, in framing an engineering organization for highway work, and in drawing the specifications for the work under it, the speaker believes that full consideration should be had, previous to arranging for the resident engineers and the inspectors, of the possibilities of securing competent "residents" for the salaries fixed to be paid; of the possibilities of securing alert and satisfactory "inspectors" for the salaries fixed for these positions; of the competency and character of the contractors likely to carry on the actual work,

and of other conditions likely to affect the decision, and then to design the machine in all its details so as best to fit all the conditions. Mr. Crosby.

It is scandalous for an engineer to design a machine requiring steel in all its parts and then, on that design, to build it of wood because wood happens to be the available material. It will be disastrous to allow the use of wood for a cog when steel is known to be needed, and every cog in the machine, even the least important, should be recognized as needing in its character an ample "factor of safety".

One word further: Such a machine should not be designed or built too delicately or too lightly. The larger and more complicated the machine, the more necessity there is for sufficient mass in the machine parts themselves to take up the shocks and vibrations, from both without and within, incident to its operation. Mass is necessary for momentum, and a sufficient momentum is desirable in this case for the purpose of continuing action in spite of the inevitable obstructions in regular and efficient operation.

FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

P. E. GREEN, M. AM. SOC. C. E. (by letter).—One of the rules adopted for this discussion is that the matter to be presented must be new. It is a very proper rule, but it should be realized that it is a very difficult one to live up to. As far as new matter is concerned, it is hard to conceive of a phase of this subject which has not been touched upon or investigated more or less thoroughly, except possibly from the laboratory end. It would appear necessary, therefore, to confine oneself to an academic discussion of the matter or to present individual experiences showing some variation from expected results. In order to bring out a broad line of discussion, it might be well, however, to systematize the factors entering into the problem and call attention to some phases which might be investigated a little more than has been done in the past, with the hope that some important new points may be presented. In opening this subject the writer will confine himself to a general outline of the problem, leaving the details to be discussed by those who follow. Mr. Green.

The subject as assigned may be logically divided into two interlocking general sections, namely: "The Selection of Materials", and "Methods of Construction". Each of these main headings may be further divided into numerous and important parts. All these parts should be taken into consideration by any public authority, such as a city, county, or State, when it is proposed to enter on a general scheme of highway improvement such as is so common at this time. In this way it is not difficult to make a logical selection of the various materials and methods involved in the construction of a system of streets or highways; and a scheme or plan of future construction

Mr. Green.

may be laid out. This does not mean that only one material shall necessarily be selected for any one street or highway. It simply means that the class or type of material shall be selected, and that choice must ultimately be made between several more or less suitable products.

A diagrammatic outline of such a general scheme covering the subject of this discussion is herewith submitted. It is not claimed that it is particularly novel or complete, but is presented for discussion and improvement. Although intended primarily for country highways, it may be used with equal facility for city streets.

DIAGRAMMATIC OUTLINE OF "FACTORS" IN ROAD WORK.

Highways under Consideration.

I.—Selection of Materials.

1.—Character of Road (City or Country).
- Trunk.
  - Through traffic road—commercial or passenger, or both.
  - Commercial road, but not necessarily through traffic.
- Secondary.
- Local.
  - Connecting important centers to trunk lines.

2.—Traffic.
- Much light fast passenger; or
- Heavy hauling; or
- Only local hauling.

Roadway widths required.

3.—Climate.
- Northern.
- Intermediate.
- Southern.

4.—Character of Supporting Country.
- Property values and benefit to be derived.
- Vicinity of large cities.
- Topography—drainage.
- Character of business using road—esthetic considerations.

5.—Materials.
- Class.
  - Superior.
  - Inferior.
    - Physical conditions governing, such as wear, crown, dust, sanitation, etc., etc.
- Cost—maintenance charges.
- Character and appearance.
- Ease of handling materials.
- Relation of material to roadway width.

II.—Methods of Construction.
1. —Laws and legal organizations governing.
2. —Location of railways with respect to improvements—railway rates.
3. —Existing good roads as affecting haul—wagon or truck.
4. —Availability of labor.

Mr. Green.

The scientific selection of the material to be used in the construction of a highway depends on all the items named in the diagram. The diagram itself looks somewhat complicated, but its use is very simple. Before commencing any large scheme of highway improvement, the outline may be studied with reference to materials, methods, etc., and simple tables prepared. Too often some essentials are not considered seriously or are forgotten, and for this reason much money, time, and labor are wasted. From time to time, somewhat similar schemes have been advanced, but frequently they deal with only one essential, such as traffic, to the exclusion of other important elements. For example, in addition to traffic, the character of a city highway should depend on the layout of the city, its topography, population, business, education, and wealth; and, further, the location of the city as regards the availability of standard materials, such as brick, wood block, crushed stone, gravel, sand, tar, asphalt, etc., etc., is vitally important.

Considering the problem of a country highway system, many entirely different factors enter into the selection of the material, such as vicinity of cities, large or small; general population and wealth of the surrounding country; whether or not the highway is to be a trunk line for through traffic, and if so, whether such traffic is light, high-speed traffic, or heavy, low-speed, commercial traffic.

If the highway is a secondary one, as regards the through traffic, it still may be a trunk line as regards its commercial traffic, such as a road leading from a trucking district to any one of our large cities. Such a highway may be comparatively a short one, yet the character of its traffic actually makes it a trunk line.

The least important class of country roads is generally referred to as "local roads". These are exactly what the name signifies, and whether or not they are to be improved depends on the wealth and education of the surrounding country and its inhabitants. Such roads are generally paid for only by local taxation.

The effect of "climate" is more important than is often considered. Some pavements, such as the various bituminous pavements, concrete, etc., are very much affected by the temperature. Thus a surface mixture of asphalt suitable for Chicago is not suitable for New Orleans; and concrete as a wearing surface should do better in Houston than Buffalo.

Three zones may be recognized: Northern, embracing the New England States, New York, Pennsylvania, Ohio, Illinois, etc.; the Intermediate, such as Virginia, Kentucky, Tennessee, Arkansas, etc.; and Southern, such as Alabama, Mississippi, and Texas, which have little winter. In the Northern States, the selection must be of materials not affected by extremes of temperature, freezing and thawing. For the Intermediate, it is not necessary for the materials to withstand great variations in temperature. For the Southern, materials which

Mr. Green. might utterly fail in the North because of inability to withstand extreme cold or great variations in temperature, may be very satisfactory. Furthermore, the exceedingly heavy and continuous rains following long hot, dry spells in the southern sections must be taken into account.

A great deal of money might often be saved by considering carefully the various classes of paving material, which may be suitable for the same purposes and yet may vary widely in cost in the different parts of the country. Thus, wood block pavement may be a very reasonable one for the business streets of small cities in regions close to the sources of supply, but for other cities, similar in size, but not so situated, it would be a grave mistake to use it, because of its prohibitive cost. This same wood block pavement has a characteristic in common with sheet asphalt: it must receive a certain minimum traffic or it may fail. On a wide roadway, having only a traffic along the center, the blocks outside this used strip will curl up under the sun and swell and "blow up" if a rainy fall follows a hot, dry summer.

Further, such a material as trap rock, though making good macadam highways in New York, probably better, as far as wear is concerned, than any of the limestone used in the West or South, would be impossible for these last locations. Yet the writer has seen specifications, for roads hundreds of miles away from granite quarries, which called for granite top macadam, though local limestone at half the cost would wear nearly as well. Again, there are sections of the country where the high cost of cement, but the comparatively low cost of good stone or gravel, makes a concrete base for a pavement quite expensive, but a crushed stone or a natural gravel rolled base would cost less and be nearly, if not entirely, as durable. Of late years, too little attention has been paid to the macadam base as a foundation for a durable pavement. The public has been educated to think that concrete is the only possible foundation. The situation is similar to that of the septic tank in sewage disposal. Many public officials have heard and believe that such a tank turns sewage into drinking water.

In those parts of the country in which there has been much road work and improvement, it is much easier to get competent foremen and good labor than in sections where little, if any, of such work has been done. The result of good workmanship is very apparent. The local officials, also, are better educated in these matters and more inclined to give an engineer sway.

It should be remembered, also, that the character of the highways, whether city or country, must depend largely on the ability of the tax-payers to pay for them. A sparsely settled region may be very desirous of having good roads and be willing to be taxed for their construction, within reason, but Wisconsin, for instance, is far less

able to pay $10 000 per mile for highways than is Massachusetts. The road mileage depends only to a limited degree on the population. Mr. Green.

There can be little question of the benefit derived from the construction of good highways, but the degree of immediate benefit should have much to do with the selection of the material for such improvement. It might be, and frequently appears, that the benefits in the long run from a thoroughly high-class permanent construction will, theoretically, far exceed those derived from any cheaper construction. Because of the large immediate benefit received from the cheaper construction, however, and because the tax-payers are unable to pay for the more expensive construction, it may be much better finance to put in the cheaper highway, and thus not over-burden the community. Future development and growth of wealth may provide equitably for the construction of a better improvement. Is it not the same principle that governed the construction of our early railroads? It is a real recognition of this principle, not altogether possessed by some engineers, that has justified in many cases the voting of long-term bonds for short-lived improvements. Such a principle must be used with caution, however. It cannot be defended in New York or Illinois, but certainly can in Texas and Oklahoma.

In addition to the various points noted thus far, the esthetic features of the wearing surface of a highway should not be forgotten. In many cases, this feature should be made a very large factor in the selection of the proper material. Thus, a smooth, well-built brick pavement might be nearly as noiseless as one of bituminous concrete, and might have a much less maintenance charge and be more sanitary; but, along a scenic highway where the adjoining country is very highly developed esthetically, such as in the vicinity of a large city where there are many great estates, it is quite probable that the bituminous concrete pavement would be considered more suitable, pleasing, and satisfactory by those who pay for it. Such a road would be used largely by automobilists and comparatively little by truck gardeners. If this highway were lined by truck farms, however, could there be any doubt that the selection, if made between the two, should be brick, especially as it has been demonstrated that the bituminous concrete does not withstand iron-tired traffic? This selection, moreover, would be the correct one, even if the pleasure traffic was very large.

One of the points which it is believed has been neglected in the past is the relation of traffic to roadway width, especially where traffic is dense. It seems to the writer that it is possible to establish a logical, if empirical, formula which would be a considerable help at times in the solution of this vexatious problem. Some years ago, in preparing a report on a popular scenic highway near Chicago, a traffic census was taken at various points, and in addition statistics on other popular

Mr. Green.

park highways in Chicago were obtained. These were plotted, together with the various existing roadway widths, and several curves tried on the same sheet. In this manner, it was finally decided that a parabolic curve having the equation:

$$Y = \sqrt{\frac{X}{6}} + 16,$$

would express the desired relation. In this equation, Y = the roadway width, in feet, and X = the number of vehicles.

Thus, if it is determined from a traffic census that within 5 or 10 years a highway will have a probable traffic of 2 000 vehicles, the formula would give a roadway width of 34 ft., as shown on Fig. 1.

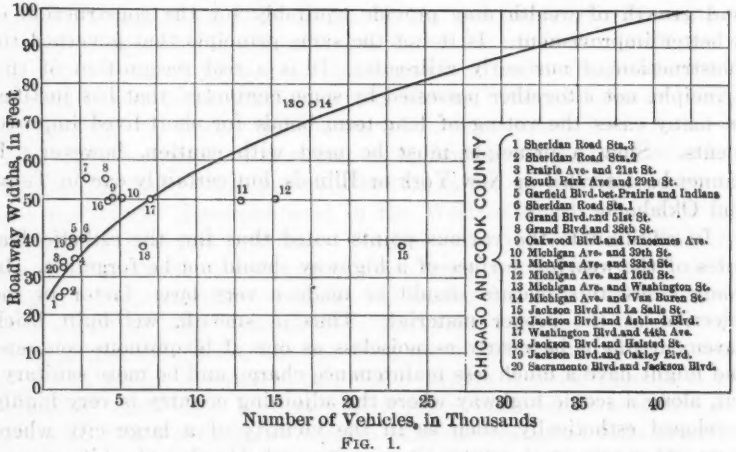

FIG. 1.

The diagram, or formula, is of course faulty as it takes into account only two factors, but it has been a help in arriving at conclusions at times, and is offered for what it is worth.

Attention would be called to the fact that, because it is decided that the roadway is to be of a certain width, say 34 ft., it does not follow that the entire road should be paved with an expensive material. Cuyahoga County, Ohio (Cleveland), has hundreds of miles of roads improved on one side with brick and on the other with plain macadam or gravel. The empty wagon and light horse traffic prefer the latter. Winona County, Minnesota, has paved the central 8 ft. of some of its highways with concrete, and provided wide macadam shoulders.

Mr. Thomes.

E. H. THOMES, M. AM. SOC. C. E.—An important factor in the selection of materials would be a standard specification for gravel, broken stone, etc., so that there would be no misunderstanding as to the sizes referred to; it would also be to the advantage of all buyers

and sellers of broken stone. At present, this material is specified by minimum, maximum, and average lengths or diameters, by passing through a ring in any one or all directions, by the diameters of the screen perforations, by arbitrary designations, and in various other ways. Some designate No. 1 stone as the first course laid, which is usually the large stone, and others designate No. 1 stone as the small stone which first passes through the stone screens. Some refer to nut size, which may be any size between those of a hazel-nut and a cocoanut. The speaker presented this matter to the Association for Standardizing Paving Specifications 3 years ago, and also in more detail to one of the road meetings of this Society 2 years ago, but very little has been accomplished since. His attention was called to this subject at the International Road Congress in London in the summer of 1913 by a report of the work done by the (British) Engineering Standards Committee on the standardization of road material. The report was published in the daily official journal of the Road Congress at London, but very little mention of that work has been made in the United States. Mr. Thomes.

The Engineering Standards Committee is supported by the Institution of Civil Engineers, The Institution of Mechanical Engineers, The Institution of Naval Architects, The Institution of Electrical Engineers, and the Iron and Steel Institute, all of Great Britain. It has issued more than sixty reports and standard specifications, mostly along mechanical and industrial lines. In the summer of 1912 a Sectional Committee on road material was formed. It recognized the need and importance of the standardization of broken stone, and that was the first matter taken up. A sub-committee of the various interests was appointed, and, after considerable discussion and investigation, the British Standard Specifications for sizes of broken stone and chippings was adopted and issued by the Engineering Standards Committee in the summer of 1913.

Stones 1 in. in diameter and less are designated as chippings. There are four standard gauges of broken stone, 1½, 2, 2½, and 3-in. Broken stone specified as 1½-in. gauge shall all pass through a 1½-in. ring, and shall consist of the following percentages by weight: Not more than 15% passing through a 1-in. ring in every direction; not less than 65% more than 1 in., and not exceeding 2 in. in greatest length by measurement; not more than 20% greater than 2 in. in greatest length by measurement. The specifications for each ½-in. larger gauge are similar to the foregoing, with ½ in. added to each dimension stated, excepting the 3-in. gauge where 4 in. greatest length is used. Six standard gauges of chippings are recommended, namely: 1, ¾, ½, ⅜, ¼, and ⅛-in. All 1-in. chippings must be capable of passing through a hole 1 in. square; and at least 70% by weight must be retained by a sieve having holes ¾ in. square. The specifications

Mr. Thomes.

for the other sizes are similar, with the substitution of the sizes of sieves. With ½-in. chippings, the sizes of square holes on which the material must be retained is $\frac{1}{16}$ in.; everything passing that sieve is to be called sand. They also specify gauge plates, sieves, and methods for testing fair samples of at least 100 lb. of the material. It will be seen that in Great Britain the broken stone is divided into more sizes than is customary or advisable in the United States. Some of the quarries there have been developed much longer; they are down deeper, and in sounder and more uniform rock. Some of them also furnish washed chippings, which are advantageous for bituminous work.

The materials and quarry practices in the United States are similar to those of Great Britain, and, with the work already done by the Engineering Standards Committee, it will be an easy matter to obtain a standard specification for sizes of broken stone. The question must be considered from the standpoint of both sellers and users, for all purposes. Conditions here seem to warrant that this subject be taken up by this Society, and in order to secure some definite action on this matter, the speaker will move that the Board of Direction be requested to appoint a Special Committee on the Standardization of Broken Stone, to report to the Society as soon as practical.*

Mr. Whinery.

S. WHINERY, M. AM. SOC. C. E.—The diagrammatic outline of factors in road work submitted by Mr. Green is generally a very satisfactory classification of the elements entering into the design of highways. The speaker, however, would remove the element of esthetic considerations from the subdivision under "Character of Supporting Country" and give it a separate sub-head at the end of the second column. It may be questionable, however, whether this item should have any place in such a diagram, devoted as it is to what may be called the economics of highways. Esthetic considerations, of course, cannot be ignored finally, but, from the engineering point of view, they are foreign to the main problem, which, in highway design, as in other engineering structures, may be reduced to the question: What design will best serve the purpose intended, with due regard to the highest yield in returns on the money invested? Necessity, of course, may force us to substitute the question: How may the highest economical results be obtained for the money available? Esthetics is not a factor in solving either problem.

Nevertheless, it must be finally considered, and the answers to the foregoing questions may have to be modified accordingly. The highway should be designed along the same lines as any other industrial

*This matter is referred to the Special Committee on Bituminous Materials for Road Construction, with the request that some action be taken to secure the standardization of broken stone in the United States.

Mr. Whinery.

structure, as, for instance, a building for manufacturing purposes. Here the efforts of owner and architect will be first directed wholly to questions of utility and economy. These settled, the question of the adaptation of the design to artistic ideas or standards will be considered, and such modifications or additions as may please the designer, or as the owner may feel willing or able to afford, will be incorporated.

In any event, the question of esthetics, however important a factor it may be considered in designing city streets, boulevards, and parkways, is usually so unimportant in as far as it relates to the roadway itself, in the case of country highways, as to be almost negligible.

The speaker cannot agree with Mr. Green that the building of temporary, inadequate highways is justifiable, except in rare instances, especially where long-period bonds are issued to pay for them. In the present well-developed status of scientific highway construction and economics, and the possibility of securing capital for enterprises of real merit, the spending of the public money for highways of temporary life and usefulness is generally inexcusable. Mr. Green's citation of early railroad history does not present a parallel case. The temporary and inadequate railroads built in the early days resulted from the undeveloped conditions of the principles and requirements involved, rather than from financial or economic considerations. Intelligent and reasonable people do not pattern after them now, to the extent of adopting an ephemeral and clearly inadequate construction, which must obviously be abandoned and reconstructed in a few years. It would be more difficult to finance such an enterprise than one properly designed to serve adequately the prospective requirements of the reasonably near future.

As a business proposition, the speaker does not believe that it is wise or justifiable to issue long-period bonds to pay for a structure the useful life of which will be but a small fraction of the period the bonds are to run. In other words, we have no ethical right to saddle upon posterity a burden from which posterity will not be benefited. From the sound business standpoint, a highway improvement the benefits from which will not redeem its cost during its life, is not a justifiable investment for a community or a State.

This, however, is not an argument in favor of unnecessary or reckless expenditure in the building of a highway. No more money should be expended on it than the conditions or needs of a community, or the general public, now economically justifies or is likely to justify during the life of the improvement. The strict application of this criterion would rule out the costly improvement of many ordinary highways where a properly constructed dirt road is all that the conditions warrant.

The considerations relating to selecting the type of roadway and the materials to be used in its construction, submitted by Mr. Green, are

Mr. Whinery.

in the main apropos and sound. It would have been appropriate for him to add a warning against the use of materials, merely because they are of local occurrence, or because of their cheapness, and a strong protest against the use of materials or methods the utility of which has not been sufficiently established by experience. The use of experimental materials and methods, except on a very small scale for trial, ought to be discouraged by the prudent engineer. Progress demands, of course, that promising innovations from established practice shall be tried out, but the conservative engineer will prefer to let other people do the experimentation, except on a scale which will not result in any considerable loss to his clients, in case of failure.

Personally, the speaker feels that, in road and pavement work, we ought to adhere pretty close to materials and methods the value of which has been clearly established.

The more important country highways are now subjected to very much the same conditions of use as the average city street. Long experience, both in America and abroad, has, after thorough trial and much foolish and often disastrous experimentation, determined that certain types of pavement and certain materials, can alone be depended on to produce satisfactory results, and their use on city streets has become standard. Among these the choice, of course, will be influenced by local conditions. Prudence and good engineering would seem to dictate that we should apply this experience and established practice to the construction of modern roads. The common argument that the first cost of a standard roadway pavement is too great to warrant its use is in most cases fallacious or untenable. An established highway is, nearly always, a permanent structure, and should be designed with reference to its true economy, that is, its value as an investment. If, for instance, a standard brick-paved roadway will last 20 years where a water-bound macadam surface will last but 5, and the former can be maintained at a less annual cost, it will be, at ordinary prices, much the more economical in the end, and the engineer should make this plain to his clients. He should vigorously discourage the common disposition of the public to waste its money on temporary highway improvements because of their low first cost, and should encourage the building of roads in such a manner as will prove most truly economical. These are such trite arguments that they seem to need to be repeated until their truth is accepted and more generally acted on.

The speaker agrees with Mr. Green that the width of the paved roadway should be determined with reference to the density of travel to which it is likely to be subjected, but thinks that the tentative formula submitted for determining the proper width, when applied to country roads, should be used with even more caution than the word tentative implies. The data on which it is based seem to have been

derived from conditions on very heavy traveled streets, boulevards, and suburban roads, where the conditions are very different from those on most country highways; but, even in the examples used, there is such a wide variation in the figures as to suggest their uselessness as a basis for determining a standard of width of roadway. Thus, as nearly as can be determined from the diagram (Fig. 1), the number of vehicles per foot of width of roadway varies from about 35 to more than 600. Mr. Whinery.

It is doubtless true that the higher figure indicates dangerous crowding, but, even so, a roadway having only 6% as many vehicles per foot of width cannot be regarded as an appropriate factor in determining a proper and safe standard for allowable density of travel.

If the proper width is to be determined by this method, only observed data, from such highways as seem to have just about the quantity of travel they can safely and conveniently accommodate, should be used. Deductions or formulas from such observations would be far more useful and trustworthy than the theoretical considerations and conclusions that some have tried to work out and apply. It is very desirable that data from highways of the character mentioned should be collected and collated.

W. W. CROSBY, M. AM. SOC. C. E.—Mr. Green is to be complimented on his interesting presentation of this subject and on his exposition of the factors involved in the selection of the materials or the methods of construction. His diagrammatic outline of factors in road work is most interesting, and if one will take the trouble to compare this diagram with that submitted by the speaker 3 years ago,* an interesting illustration will be afforded of the ideas in common as well as those of difference which may be held by two individuals. Mr. Crosby.

The speaker has been particularly interested in the scientific solution, proposed by Mr. Green, of that troublesome problem, the determination of the proper width of a roadway, and desires to express his belief that it is this sort of treatment of many of the present questions in highway engineering that will do most to advance the status of this particular branch of the profession and to erect for its future growth a substantial foundation.

In reference to this matter, and with the hope of furnishing data for the basis of such treatment of some of the problems connected with the selection of materials and methods in highway work where the traffic factor seems preponderating, the speaker offers the following concerning traffic records in general and in a particular instance coming under his observation:

In 1903 he urged highway engineers to take traffic censuses and study the relations between traffic conditions and the life, or ex-

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 6.

Mr. Crosby.

pense for maintenance, of road crusts. He also suggested that from the study of such relations much might be learned that could be used advantageously in making selections for construction under known or determinable traffic and other conditions.

The Illinois Highway Commission began this traffic census in 1906. In 1909, the Massachusetts Highway Commission took a census of traffic on Massachusetts roads.* Other traffic censuses have been taken and reported by officials and individuals from time to time since these dates, and though at first there was some skepticism as to the value of such investigations, the speaker believes that there is now a fair amount of general agreement that, with proper deductions made therefrom, they are of great value. He also believes that there is general agreement now that a road crust satisfactory and economical for sustaining rubber-tired motor traffic may not be so, either for horse-drawn or for hard-tired traffic, and *vice versa*, as he has stated previously.

During the summer of 1913, in order to permit the improvement of a highway by the State Roads Commission of Maryland, the Board of Park Commissioners of Baltimore opened to all kinds of traffic a park road paralleling the Reisterstown Turnpike for a distance of 2 500 ft. Previously, this park road had been kept in excellent condition by the maintenance, on the quartz-schist macadam, of a bituminous (or "pitch") surface, at an annual cost of about 9 cents per sq. yd. Within 90 days after the opening of the road to all kinds of traffic, this surface or "carpet" became in dry weather a dusty mass of loose material and in wet weather a mess of slimy black mud an inch or more thick. Depressions in the macadam beneath also began to show soon afterward, and by December, 1913, the road crust appeared to be generally going to pieces. The quartz-schist had shown a rattler test of about 4, though its cementation power, in use, had proved to be high—probably due to its iron content.

Southward from this 2 500-ft. stretch, the pleasure traffic continued along the main road through the park, the commercial traffic, which consisted mainly of horse-drawn, hard-tired vehicles and motor trucks, being turned out of the park at this point. The traffic on the 2 500-ft. stretch before its opening to commercial vehicles was therefore equal to that now using the road southward from this stretch, as shown by the census under "B" in Table 1; the combined traffic on the 2 500-ft. stretch is that shown under "A".

The carpet referred to in this case was made up by successive applications, in 1911, 1912, and 1913, of "75% Asphalt Oil", the layer of each being covered with pea gravel or stone chips, and at the time of its first subjection to the new traffic it was about ¼ in. thick on top of the macadam. The last application of bituminous material (or "pitch compound") and stone chips was made the day before opening the

* Report, Massachusetts Highway Commission, 1909, p. 128, etc.

road to mixed traffic. The width of the carriageway, exclusive of gutters, is 22 ft. Mr. Crosby.

The figures of the traffic census taken in 1904 are inserted in Table 1 in order to give an idea of the increase or change in character of the traffic between that time and the present. They also give some information as to the amount of traffic under which a water-bound macadam of comparatively soft broken stone may be maintained satisfactorily at a not excessive cost. Such a road crust in this instance required treatment with a pitch compound before the traffic reached the figure in "A", in order to permit its maintenance with satisfaction at any cost.

TABLE 1.—TRAFFIC CENSUS ON MAIN ROAD IN DRUID HILL PARK, BALTIMORE.

Classification	Maryland factor.	November 24th to 30th, 1904.		November 20th, 1913.			
				"A"		"B"	
		Average per hour.	Units.	Average per hour.	Units.	Average per hour.	Units.
Saddled horses	1	3	3	10	10	0	0
One-horse vehicles	2	28	56	25	50	7	14
Two " "	4	10	150	35	140	0	0
Three " "	6			1	6	0	0
Four " "	8			0	0	0	0
Six " "	12			1	12	0	0
Total horse traffic		41	209	72	218	7	14
Bicycles	2	2	4	8	16	6	12
Motor runabouts	10	0	0	9	90	7	70
4-seat motor cars	20	1	20	43	860	39	780
6 " " "	40	0	0	37	1 480	35	1 400
Motor trucks	20	0	0	9	180	0	0
Totals		44	233	178	2 844	94	2 276

In making deductions from the data in Table 1, it may be of interest and value to refer to the similar data and the conclusions thereon,* furnished by the speaker a year ago concerning the Park Heights Avenue experimental work.

One further point may be indicated. It will be noted from the traffic tables that the increase in traffic on this section, due to the admission of commercial vehicles, was 204 units of horse-drawn and 364 units of motor traffic, the latter being composed mainly of motor trucks. Both these types have a high abrasive or crumbling effect.

* *Proceedings*, Am. Soc. C. E., for September, 1913, p. 1702 *et seq.*; or *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 188 *et seq.*

Mr. Crosby. Evidently, the thin carpet over the soft macadam stones was unequal to the task of maintaining its integrity under this traffic, or of absorbing the shocks coming on it sufficiently to prevent the disintegration of the stones under it. Hence, under this traffic, the stones were abraded or broken to pieces, the bond of the carpet to the stable foundation was thus destroyed, and the carpet itself aided in going to pieces. The actual destruction proceeded along these lines, the carpet first scaling off in spots and revealing in these places a ground-up condition of the macadam surface beneath. Then the scales of the carpet, thus loosened, disintegrated, and finally the whole carpet went into dust or mud.

The experience here, as well as in some other cases with similar conditions coming under the speaker's observation, is quite similar to many instances of the failure of thin carpets on concrete roadways, and supports the theory that, with friable bases, the carpet must have such thickness and character as will prevent the disintegration, under the shocks of traffic, of the surface of the base itself, if successful maintenance is to be had.

Mr. Brooke. MARK BROOKE, ESQ.* (by letter).—There is no one best paving material for all conditions, and the necessity of carefully considering in each case the many factors shown with such definiteness and detail in Mr. Green's diagram cannot be emphasized too much. As Mr. Green has stated, the use of that diagram in practice is comparatively simple, for it will be found that the choice in many cases will be restricted either by limitations of first cost or by local or general conditions which automatically cut out certain materials from consideration.

In contradistinction to the bridge engineer, for example, the highway engineer often finds that the sum of money placed at his disposal is not based on his own estimate of a definite project, but that he is allotted, in a general distribution of funds based on anything except a careful estimate or budget, a lump sum of money which he must spread over a certain section to the best advantage of the entire area. The question is then, not what is the best material, but how much he can afford to spend on a given street or road, so that the problem is apt to resolve itself into a selection of materials for the most suitable pavement which can be constructed for a definite limited sum, say, for $1.25 per sq. yd. Even in this narrow field of choice careful consideration should be given to all the items in the diagram, as far as the limitation of cost will permit.

Again, it will be found possible to simplify the selection of materials by applying once for all such of the factors in the diagram as have a general and permanent application in any given locality, thus elimi-

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nating a large class of materials, and restricting the choice to a few types. Mr. Brooke.

In Washington, for example, as a result of experience and of consideration of the various factors such as first cost, character and volume of traffic, suitability of local material, esthetic considerations, etc., the problem has been reduced to a selection of one of three types of asphalt pavement for city streets, and bituminous concrete, cement concrete, or water-bound macadam with a surface treatment of light tar or oil, for suburban streets and county roads.

In the city practically nothing is used but a standard sheet-asphalt, a very limited quantity of 2-in. asphalt block, and an increasing quantity of bituminous concrete in accordance with the specification of the department.

This bituminous concrete, which is very similar to the various pavements of that type laid between 1870 and 1880, appears to have all the advantages and but few of the defects of sheet-asphalt, and is cheaper.

A gravel cement concrete base is now being used under all pavements, except in a few instances where old macadam roads have been surfaced with bituminous concrete.

Macadam, especially an old macadam road, will make an excellent base which will outlast any bituminous wearing surface yet devised, but it cannot be kept free from local settlements, nor can it be easily or economically resurfaced. The man who lays bituminous concrete on a macadam base should realize that he is storing up trouble for himself or his successor 15 years hence.

Whether or not it is advisable to invest in the more expensive base to reduce the cost of future resurfacing is, of course, a question every one must decide for himself.

The engineers in Washington for some time past have been of the opinion that it is better, as a general practice, to lay a base which will not have to be destroyed every time a new surface is put on it, and hope to relieve their successors of the extra expense and difficulties encountered during the last 10 years in resurfacing pavements laid on macadam and bituminous macadam bases—types of construction which conformed to the best engineering practice of the time, when the use of Portland cement was still in its infancy.

For 30 years a concrete base has been laid without either longitudinal or transverse joints, with no significant bad results, and the writer doubts very much the necessity of the elaborate and costly provisions for such construction which are so often seen in cement road work. On some of the cement pavements in Washington, no joints whatever have been provided, on others there is a thin transverse joint every 50 ft., made by bringing the concrete up to a vertical face, laying a strip of folded tar paper against this face, and then depositing the

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next batches against the paper. Thus far, no difference in the action of the two classes of construction has been observed. If any joint is used it should be as thin as possible. A wide joint is a weak joint, and its partial contraction will invariably force the filler into a pronounced ridge, as objectionable to motor traffic as any thank-you-ma'am.

When this pavement is laid with a curb it is kept about 1 in. below grade, so that, should the thin bituminous carpet with which it is covered prove an uneconomical form of wearing surface, the pavement can be used as the base for a thicker surface of bituminous concrete and still have sufficient face of curb showing.

For many years trap rock has been the standard material for water-bound macadam. Recent experiences in the oil and tar surface treatment of trap and limestone roads indicate that, for moderate traffic at least, the surface treatment does as well with the limestone as with the trap, and the saving effected by using limestone about offsets the first cost of the surface treatment.

Referring to the practice mentioned by Mr. Green, of paving the center 8 ft. of certain roads with concrete and providing wide macadam shoulders for the light traffic, the writer believes it is a mistake not to make the paved strip wide enough to carry two lines of traffic. If this is done, it is unnecessary for passing vehicles to cross the joint between the concrete and the shoulders—the weakest part of the road—a condition which would be particularly objectionable at seasons when the earth shoulders were unsuitable for traffic, and all vehicles would seek the paved portion of the road. The engineers in Washington have had this experience in the case of a trunk-line road paved with granite block, the center 8 ft. of which was laid with asphalt for the benefit of light traffic. As a matter of fact, all traffic, both slow and fast, tries to get on and stay on this 8-ft. strip, thus increasing both the congestion of traffic and the wear on the asphalt.

The absence of any numerical weights assigned to the factors in Mr. Green's diagram appeals to the writer as being most sensible. The relative values to be given these factors in selecting a material must be different for each case, and a table which attempts to fix values for general application is apt to be misleading.

Materials and methods successful in one case are failures in another, and the lack of knowledge or realization of the difference in conditions in the two cases often leads one to draw general conclusions which are not warranted.

To attempt, with present knowledge, to solve road problems by the use of general formulas or tables is, as a well-known highway engineer has described it, "simply holding at arm's length work which should be brought close to us".

By careful study and accurate records of failures and successes, by standardization of specifications and tests, and by uniformity in

methods of taking traffic censuses, one can largely eliminate the elements of personal taste and judgment in the consideration of the various factors under discussion, and still further simplify and standardize the selection of materials. Mr. Brooke.

EQUIPMENT AND METHODS FOR MAINTAINING BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

WILLIAM R. FARRINGTON, M. AM. SOC. C. E. (by letter).—The maintenance of bituminous surfaces and bituminous pavements is divided naturally into three stages. The first includes what is usually classified as ordinary maintenance, such as patching, covering, etc. The second covers re-treatment of the surfaces and the renewal of the seal coats on the pavements; and the third includes the re-shaping or re-surfacing of roadways carrying bituminous surfaces, and the re-surfacing or reconstruction of the pavements. Mr. Farrington.

The writer will discuss the work with which he is familiar, on roads outside the cities and large centers, and will consider the equipment needed and the methods to be followed in maintaining bituminous surfaces consisting of hot or cold oil, or tar, and sand, gravel, or chips; also, bituminous pavements composed of broken stone and tar, oil or asphalt; or of sand and oil.

It is not intended to take up the cost of maintenance, for though this cost might be regarded as a criterion of suitable equipment and proper methods, it probably varies so much in different localities that, after all, a comparison of costs is not of much value in a general discussion, unless the conditions are well understood and the prices paid in any locality are known.

It is assumed that maintenance work will usually be handled by a department with its own gangs and equipment, rather than by contract, except possibly in case of extensive re-surfacing.

Patching.—For patching, only a small gang is needed, and the equipment is simple. There should be one or two portable melting kettles, a small portable tool-house, and one or two small tents for the laborers, unless a van is provided.

The methods to be followed in patching the different kinds of surfaces and pavements are much the same. Generally, one should use the same kind of bituminous material for patching as was used in forming the surface, and ordinarily it is not advisable to select a much heavier grade, except with stone, slag, or chips, for filling depressions.

In the writer's opinion, only the lightest, or so-called dust-laying, oils or tars, should be used without the addition of some grit; and, even with these, a light covering is probably economical. If applied

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without covering, however, patching is usually not advisable, as the cost of an additional application is small.

In patching an ordinary oil or tar surface, one should first attend to the depressions. Slight depressions can be filled with oil, or tar, and sand, gravel, or chips; but if they are deep, stone or slag should be added. The depressions should be cleaned, and the edges cut out if necessary, in order to allow a thorough bonding of the new and old material.

Although the best results are obtained by mixing the bituminous material and sand, or stone, etc., before applying, satisfactory results may be obtained without mixing, if the work is done carefully. In filling slight depressions, where the materials are not mixed before applying, a thin coating of oil or tar is spread over the bottom and covered with sand, gravel, or chips. If stone or slag is used with oil for filling deep depressions, the stone or slag is spread over the oil with which the bottom has been coated, then more oil is applied and covered with grit. If tar is used, the methods are similar, but more bituminous material is required.

After filling the depressions, any bare places, or places where the bituminous surface is practically worn through, should be coated, the surface first being cleaned and grit being spread over the oil or tar. In no case should much more than ½ gal. of bituminous material per sq. yd. be used for coating the surface, as any surplus oil or tar will result eventually in the rolling of the surface and the formation of bunches.

Most of the so-called cold tars may be applied without heating, as may the lighter grades of the so-called cold oils, or road oils; but the heavier grades must be heated in order to give the best results in patching.

A much heavier oil or tar than that used in the surface is inadvisable for patching, yet a somewhat lighter grade may be used. Excellent results are obtained by patching a hot oil or tar blanket with the heaviest cold oil or tar, and with these there is much less chance for the formation of bunches than with the heavier materials.

If a pavement composed of stone and bituminous material is constructed properly, not much in the way of maintenance should be required for some time, but depressions will develop eventually, and some patching will be needed. In patching bituminous pavements, including those composed of sand and oil, the methods are similar to those for a bituminous surface, except that, with the sand and oil pavements, deep depressions should be filled with heavy oil and sand, properly heated and mixed before applying; or, if the heavy oil is not available, or there are no facilities for heating the sand, an oil similar to that used for surface work will give good results with slag or cinders, without mixing.

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Covering.—On any bituminous surface, and especially one in the formation of which oil is used, more or less covering with sand, gravel, or chips will be required, either to take up free oil or tar, which will work to the surface for some time after it is applied, or later to prevent the picking up of the surface, which may soften under certain conditions. Covering is also partly a remedy for the slippery condition incidental to any bituminous surface during cold weather.

Re-Oiling or Re-Tarviating.—For re-treating a bituminous surface, practically the same equipment is required and the same methods should be followed as for the original treatment, except that usually less bituminous material is needed.

The life of a bituminous surface, or the length of time elapsing before re-treatment is necessary, will depend on local conditions, and the amount and kind of travel, etc. Where conditions are favorable for such treatment, however, and the travel, especially the horse-drawn travel, is not too heavy, a bituminous surface consisting of grit and about $\frac{1}{2}$ gal. per sq. yd. of the heaviest hot oil suitable for blanket work should last from 3 to 5 years without re-treatment, if patched occasionally and otherwise maintained properly. In re-treating such a surface, not more than $\frac{1}{4}$ gal. of oil per sq. yd. should be used, unless practically all the original surface is gone; and it is not economical to delay re-treatment until this point is reached, as without the protection of the bituminous surface, the road metal will tend to ravel and the roadway to disintegrate.

A bituminous surface formed by the application of from $\frac{1}{8}$ to $\frac{1}{4}$ gal. of light oil per sq. yd., with a small quantity of covering, will last at the best not more than one season, and it will probably be necessary to cover such a surface quite heavily in the fall in order to carry it through the winter and until another application is made the following year. Where from $\frac{3}{8}$ to $\frac{1}{2}$ gal. per sq. yd. of the heaviest cold oil is used with grit, re-treatment with about the same quantity of oil will usually be necessary the second year; but, after that, the application of from $\frac{1}{8}$ to $\frac{1}{3}$ gal. per sq. yd. per season should be sufficient, and an application may not be necessary each year, although, if omitted, the patching will naturally be increased.

If refined coal-gas tar is used, about $\frac{1}{2}$ gal. per sq. yd. is required during the first and second seasons. After that, with cold tar, a $\frac{1}{4}$- to $\frac{1}{3}$-gal. application will usually be necessary each season, although the yearly application may be omitted occasionally. With hot tar, ordinarily a $\frac{1}{4}$- to $\frac{1}{2}$-gal. application should be made every second year. Where unrefined water-gas tar is applied, two $\frac{3}{8}$-gal. applications are advisable each year; but, if only one $\frac{1}{2}$-gal. application is made, a considerable quantity of patching will be required.

As to equipment, for handling the lighter grades of bituminous materials, one or two distributor carts and a pump for transferring

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these materials from the tank-cars are required, and there should be a tool-house, also tents for the laborers, unless vans are provided.

For handling the heaviest grades of oil or tar used for surface work, the following plant is needed:

One 10- or 12-ton steam roller,
One portable heating boiler,
One steam pump,
Two steel tank-wagons,
One pressure distributor (if mounted separately, or two if attached to the tank wagons),
One or two watering carts,
One 2-horse sweeper,
One portable tool-house, and
Two large tents or vans for laborers.

As to distributors, for handling the road oils or the corresponding grades of tars, a gravity machine may be used with good results, if the bituminous material is broomed after it is applied; but the cost of distribution will be less and the results more satisfactory if a pressure distributor is used. For this work, a distributor cart with a pump mounted on a frame back of the tank, and connected by chain with a large sprocket bolted to one of the rear wheels is most satisfactory. Under favorable conditions, good results have been obtained with gravity machines in distributing even the heaviest grades of bituminous materials for surface work; but the chances of failure, if the conditions are not favorable, are such that they should always be applied by pressure distributors if possible. The distributor just described, or one with a gas-engine pump mounted either on the tank-wagon or on separate gear, may be used, but one operated by steam will give the best results. One advantage of the steam distributor is the possibility of cleaning out the pipes and nozzles by blowing steam through them. Of the steam distributors, there seems to be little choice between the one using the direct pressure of steam in the tank for distributing and the pump type, each having its advantages.

To facilitate covering, the sand or other material should be piled in advance on the side of the road.

Before commencing to distribute the bituminous material, the surface should be cleaned with a horse sweeper, and with hand-brooms, if necessary. Any depressions should be filled and the surface patched as needed.

Where the heavier grades of oil or tar are used, it is well to water the old surface before distributing the new bituminous material, but watering is not necessary with the lighter grades unless the surface is unusually dry.

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It is advisable to cover the bituminous material lightly at first, applying more grit as needed, for though some bituminous material will take up only enough grit to form a good mixture, without reference to the quantity put on, in other cases, if there is surplus covering, the oil or tar will continue to work up until the mixture is weakened to such an extent that the best results cannot be obtained.

Seal Coats.—Although a seal coat or wearing coat may not in all cases be necessary on pavements composed of broken stone and bituminous material, the writer believes that it is economical and should usually be applied, either in connection with construction, or later, except possibly where asphalt is used in the pavement. The equipment and methods for applying or renewing such coats are similar to those for the re-treatment of the surface; but, unless asphalt is used, the bituminous material for the seal coat should be of a lighter grade than that incorporated in the pavement. Oil may be used with good results on a tar pavement, and is advantageous, in some cases, in that it is less slippery than tar and also acts to some extent as a dust absorbent.

Re-Shaping or Re-Surfacing.—Although a surface consisting of grit and suitable bituminous material will carry ordinary travel and relieve the road metal of much of the wear, it alone is not a permanent form of treatment. As a result of the gradual wear of the road metal, and because repeated treatment and patching with bituminous material will cause the formation of bunches and an uneven surface, re-shaping or re-surfacing of the roadway will be necessary, eventually.

When this stage is reached it is always well to consider a more permanent form of pavement; for it is an established fact that a bituminous surface will not carry a large amount of horse-drawn travel, and will fail under certain conditions which the writer believes are not as yet clearly understood; and although a bituminous surface may have been satisfactory in a certain locality in the past, the conditions may change or the travel increase largely.

It is assumed, however, that there will not always be money available for the construction of a bituminous pavement, in which case the alternative is the re-shaping or re-surfacing of the roadway and the application of another bituminous surface.

The equipment needed for re-shaping or re-surfacing work, to be done in connection with re-oiling or re-tarviating, is the same as that for re-treatment, except that two steam rollers will be needed, also a straight-tooth and a spring-tooth harrow.

If stone is to be crushed, there should be a portable crushing outfit.

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In re-shaping or re-surfacing a roadway with a bituminous surface, the old surface should first be thoroughly broken up with the roller picks or a scarifier and removed with stone forks.

The loosening of the road metal, partly accomplished by the roller picks or scarifier used in breaking up the surface, should be completed with a straight-tooth harrow, the use of which also tends to shake the worn out material to the bottom. This should be followed by a spring-tooth harrow, which, because of the curved teeth, will bring the stone or larger particles of road metal to the top. The surface should be shaped with rakes, any new stone or other road metal needed should be put on, and the road watered and thoroughly rolled. If any considerable quantity of new stone, etc., is used, the voids should be partly filled with sand, stone screenings, or chips; but, if only a small quantity of new road metal is put on, there will usually be sufficient fine material in the road to bond it.

Either a macadam binder or the heaviest road oil may be used, but, ordinarily, the former will give the best results. About ½ gal. of oil per sq. yd., should be used in two applications, with a covering of grit after each.

Ordinarily, it is not advisable to use tar for such re-shaping or re-surfacing work, unless a bituminous pavement is to be formed by penetration. In this case, usually more new stone or other road metal is needed, and the voids for about 2 in. below the surface should not be filled with fine material. For grouting 1¼ gal. of heavy tar should be used, and a seal coat should be formed by the application of about ½ gal. of a lighter tar with grit; or, if preferred, asphalt may be used instead of tar.

The foregoing description of maintenance work applies to a bituminous surface on a roadway of broken stone, slag, or screened gravel, rather than to such a surface on unscreened gravel; but though the methods for handling a surface on the latter form of roadway are somewhat different, mention can only be made of the fact that although, for the first application, one should use a fairly light grade of bituminous material which will penetrate somewhat below the road surface and thus form a real bond between the bituminous material and the gravel, heavier grades of oil or tar may be used, with good results, for later treatment.

Re-Surfacing of Bituminous Pavement.—In a discussion on maintenance, it does not seem advisable to devote much space to the re-surfacing of bituminous pavements, because such work is probably better classified as reconstruction than maintenance, but attention is called to the fact that the life of pavements composed of broken stone and bituminous material is as yet problematical on roads outside of cities

and large centers, because such pavements have been in use, under these conditions, only a few years. Mr. Farrington.

Storage and Repairs.—Incidental to a consideration of equipment is the question of proper care and up-keep of the outfits. There should be at headquarters, a suitable storage place, with sufficient equipment and proper facilities for making ordinary repairs. It is not only advantageous to have the repairs made by or under the supervision of those responsible for the apparatus during the working season, but, by having the work done by men from the regular force, it is possible to keep the most competent men busy during the winter.

If the amount of work done warrants the employment of a storekeeper, there should be kept at all times, at the storage place, a sufficient stock of tools, etc., to supply the needs of all the gangs.

Reports and Accounts.—It is not possible, in the time at the writer's disposal, to consider in detail the question of the reports, records, and accounts which should be kept on maintenance work. The writer believes, however, that although a consideration of cost is not of especial value in a discussion covering work in widely separated localities, a comparison of the cost of work in different localities in one section is valuable, and is necessary in order to determine what equipment and methods are most economical, and also whether the work is handled efficiently.

Daily reports giving details of the labor cost should be submitted by the foreman, and a record of these, as well as of bills for materials, supplies, etc., should be kept in such a way that the cost of the different kinds of work can be easily ascertained at any time.

H. B. PULLAR, ASSOC. M. AM. SOC. C. E. (by letter).—The methods of maintaining bituminous surfaces and pavements are unquestionably of great importance. Mr. Farrington has covered very fully the matter of maintaining bituminous surfaces and bituminous roads constructed by the penetration method. He has stated that he does not consider it advisable to devote much time to the re-surfacing of bituminous pavements. Although it may not be advisable to devote much time to the total re-surfacing of bituminous pavements, much more might be said regarding their maintenance and repair. The method and equipment necessary for repairing sheet-asphalt and asphaltic-concrete pavements in small towns and cities have been of great importance, and it is on account of the fear of the cost and trouble in making satisfactory repairs that many small towns have given preference to brick, block, and other types in place of bituminous pavements. If proper care is taken and proper equipment is secured, there is no more trouble, nor any greater expense, in repairing bituminous pavements satisfactorily than with Mr. Pullar.

Mr. Pullar.

any other kind. Methods and equipment necessary for repairing these pavements will be discussed later.

It has only been within recent years that much consideration has been given to the repairing of sheet-asphalt pavements. Until recently, the only thing which could be done was to cut out the bad part of the pavement and replace it with new. On streets where the total repairs are more than 50 or 60% of the pavement, this is still the best method. Where this plan is used, all the bad material should be cut out, the edges of the old pavement should be carefully painted with the pure bituminous cement, and then the new material should be put in by a method similar to that used in constructing the pavement. As Mr. Farrington states, an effort should be made to make the repairs with materials of the same class as those used in the original pavement. In the larger cities, or wherever it is possible, the most economical way to obtain the mixture is from some paving contractor. In smaller cities it is advisable to obtain one of the small mixing plants which are on the market and now available. The "Rapid" mixer has proved to be of exceptional value for this work, and is now used by a number of cities in the Middle West for repairing both sheet-asphalt and asphaltic-concrete pavements.

For sheet-asphalt pavements, in which only small depressions or waves appear, the most economical and probably the best method is to burn off about ½ in. of the top and make the repairs by what is known as a "skin patch". This method has been used very successfully in various parts of the country, and there are a number of excellent burners on the market. The burning can be done at about the same expense as would be involved in ripping up the pavement, and in this case it is only necessary to put on a patch about ½ in. thick, instead of cutting out and replacing the entire thickness of the pavement.

The "Lutz" heater method for repairing is based on the same principle—burning off a small quantity of the top surface and replacing it with a "skin patch". The surface is heated by a hot-air blast, and claims are made that no injury is done to the pavement, with the exception of the upper ½ in. which is removed. This heater has been used successfully by a number of the larger cities for re-surfacing work, and it is possible to repair about 500 sq. yd. per day with one machine. Care and experience, however, are necessary to repair or re-surface pavements properly by burning. It should not be attempted by anybody without some knowledge of the proper method. It is not a method which can be recommended for general use in small cities or towns, or on public highways.

During the past few years many attempts have been made to remelt and use the old material for repair work, but, up to the present time, no successful method has been developed. The Noyes crushing machine has been used successfully for pulverizing the old material,

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but the reheating has proved a difficult matter, and thus far has not met with success. The material, after being pulverized by the Noyes crusher, can be used to a considerable extent in a binder mixture, and this idea is being adopted in a number of places. Some of the small mixers claim to be able to re-melt old material satisfactorily, but in nearly every instance considerable burning takes place.

About 2 years ago the writer made a new experiment in repairing sheet-asphalt pavements. The section of pavement to be repaired was very badly cracked, and contained three holes, each about 1 sq. yd. in area. The holes were first cleaned out and the edges trimmed. Stone, ranging from ¼ to 1 in. in diameter, was then placed in each hole and tamped, and to this there was applied about 1 gal. per sq. yd. of a heavy asphalt binder. Chips were then spread over the surface and thoroughly tamped into the voids. Then a squeegee or seal coat was applied over the entire surface, including the stone patches. A recent inspection of this small strip of pavement showed that it was still in excellent condition, so that, without question, this method has already prolonged the life of that particular piece of pavement 2 years, and from indications, will continue to prolong its life another 2 or 3 years. On account of the success of this experiment this method has been used for repairing sheet-asphalt pavement in two or three other small cities in various parts of the country, and up to the present time, reports have been very favorable. This method could be used very satisfactorily in small towns and villages, and would be much more economical than the use of brick or cement for repairing bituminous surfaces.

In repairing asphaltic-concrete pavements of various types, the best method is to use a small mixing plant, such as the "Rapid" mixer, or other small mixer of similar construction. The nominal cost of these machines and the ease with which repairs can be made, will probably result in their more general use for repair and maintenance work.

There are two principal causes for trouble with bituminous surfaces consisting of the superficial treatment by use of tar, oil, or asphalt. One is the fact that, in a number of instances, too much bituminous material has been used. This produces a pavement which is very apt to rut or become wavy, and it is also very disagreeable to the eye. The other source of trouble is due to lack of sufficient bituminous material, or too great a length of time intervening between treatments. If the surface is carefully inspected at frequent intervals, and treated at the proper time and with the proper quantity of material, the road or pavement can be kept in excellent condition at a small cost.

The writer agrees with Mr. Farrington in the statement that it is not advisable at present to discuss the cost of maintenance, as this

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varies considerably in different parts of the country, and, to a great extent, depends on local conditions. Cost data would be valuable even when work in different localities is considered, provided there was a standard system for giving cost, and one which would take care of different units. The total cost per square yard is of little importance unless unit costs are given. A standardization of a system for giving cost data, in both paving and road construction, and in paving and road maintenance, is greatly needed, and much valuable work could be done by a committee of engineers appointed for this purpose.

The question of equipment and methods for maintaining bituminous surfaces and bituminous pavements, is a very broad one, and it is impossible to cover it fully in one discussion, but it is hoped that further experience may be cited on this same topic, especially in reference to a cost system and to the best methods for use in small towns and villages.

Mr. Blanchard.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Mr. Farrington properly considers the work of maintaining bituminous surfaces and bituminous pavements under three heads: first, routine maintenance, being the continuous repair of small areas of a surface or pavement; second, re-applications of bituminous materials over practically the whole surface; third, reconstruction of the wearing course.

The second and third classes of maintenance may be accomplished by day labor or contract, or a combination thereof, the problems involved being usually identical with those of primary construction. The relative advantages of the general methods referred to and the plant equipment required have formed the subject-matter of several previous discussions before this Society.* The speaker will confine his discussion to the field of maintenance work first mentioned above.

In considering the equipment required to accomplish continuous maintenance of bituminous surfaces and bituminous pavements, it seems desirable to cite certain factors and principles which appear to be essentials of economical and satisfactory maintenance.

1.—Small failures of the wearing course, and the wearing away of the bituminous surface on comparatively small areas should be repaired immediately. Otherwise the rate of disintegration will be materially increased, the annual maintenance charge will be greater than necessary, and the surface will be unsatisfactory to the users of the highway.

2.—Continuous maintenance should be conducted so that the highway may be at once opened to traffic without injury resulting to the repaired sections. For the repair of potholes, ruts, and other holes or depressions, this implies the use of bituminous materials of much lower penetration than have ordinarily been used in many cases. If

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 25; Vol. LXXV, p. 548; and Vol. LXXVII, p. 171.

practicable, the same type and grade of bituminous material should be used for the surface application as was placed in the original construction. Mr. Blanchard.

3.—In order to accomplish satisfactory results the labor should be efficient and should be under the supervision of an engineer experienced in the construction and maintenance of all types of highways, and with all kinds of bituminous materials used in the area under his jurisdiction.

4.—The intimate relationship between the methods and materials used in construction and the character of the maintenance, in the case of bituminous surfaces and bituminous pavements, is such that, to ensure success, the construction and maintenance of highways in a unit of area, or a certain mileage, should be under the supervision of one engineer.

To accomplish continuous maintenance economically and efficiently under the foregoing conditions, the flying squadron, operated under the direction of an engineer in charge of construction and maintenance of highways in the area covered, appears to be the logical solution. The equipment of such a squadron will naturally depend on local conditions, such as the mileage of highways to be maintained, their relative location, the types of surfaces and pavements, the kinds of bituminous materials used, the mileage under guaranty, etc.

As an example of a definite problem in continuous maintenance, take a county or a division of a State where a considerable mileage of bituminous surfaces and bituminous pavements has been constructed. Granted a well-maintained system, the work to be accomplished by a flying squadron would consist, first, of routine repairs, including filling all holes, ruts, and other depressions with bituminous concrete, using a type of aggregate and bituminous material suitable to each case, and, second, applying bituminous materials to all areas which gave indication of being in such condition that they should be re-treated at once rather than wait until the whole surface required another application. It will be practicable in many cases for the flying squadron to repaint guard-rails and perform other routine repair work. Under such conditions, it is believed that the use of a motor truck properly equipped will prove most satisfactory. The suggestion to use a motor truck in repair work on highways is not novel, trucks having been recommended and used for this purpose for several years.

The equipment to be carried by the motor truck for the special repair work outlined should include the following machinery and tools: a rotary heater and pug mill mixer; two heating tanks; a surface heater; storage tanks and barrels for bituminous materials of different types and grades; storage capacity for small tools such as brushes, squeegees, tampers, cutters, pouring cans, irons, shovels, picks, and hoes; and storage capacity for paints for guard-rails, mineral aggregates, and road metal of several sizes. Although a storage capacity

Mr. Blanchard.

for mineral aggregates and road metal is called for in this equipment, it should be noted that only a few cubic feet of these materials will be carried, because the maintenance work performed will be confined to the repair of small patches. Usually, it will be practicable to reload a truck each day. The fixtures for the equipment described should be arranged so as to be readily removed, thus allowing the truck to be utilized for general hauling purposes. The equipment recommended should not be confounded with the elaborate equipment designed to cover all phases of repair work recommended by some engineers. The speaker believes there is danger in overloading a truck with accessories and thus rendering its operation uneconomical.

The equipment recommended herein could be carried by a 5-ton truck, capable of traveling 12 miles per hour, and having a gasoline storage capacity for a run of 100 miles. A clear body of 7 by 15 ft. would be available for the installation of the equipment. The cost of operation during an 8-hour day would vary from $10 to $20, covering wages of chauffeur, rent of garage, interest on first cost, maintenance charges, depreciation, insurance, tires, gasoline, oil, and grease. The speaker is indebted to Mr. Alfred F. Masury, Service Manager, The International Motor Company, for details relative to truck operation.

Mr. Fulweiler.

W. H. FULWEILER, ASSOC. M. AM. SOC. C. E. (by letter).—It is not economical to apply a bituminous material without some covering, as the greatest value of the treatment lies in the added wear given to the surface of the road by the mineral aggregate used as a covering, which the bituminous material merely serves to protect and retain in position. It is always economical, especially in the re-treatment of bituminous surfaces, to repair the surface before applying the bituminous treatment.

In patching, it is very important to see that the edge of the patch is cut back slightly, so that it not only forms a square edge for the new material, but that the material on the edge of the patch, which is invariably partly disintegrated by the action of the water, is removed.

The writer would like to inquire whether Mr. Farrington has any explanation for his statement that "Where tar is used in patching, more of it is required than with other materials."

As to the quantity of material to be applied on the surface, the writer's experience indicates that should it be desired to apply ½ gal. as a re-treatment on a bituminous surface, the best results would be obtained by applying not more than ¼ gal. at a time—with just sufficient covering to enable the wagon or motor truck to pass over the surface—then applying the remainder of the material and adding the stone used for covering. Where such a large quantity of material (½ gal.) is used in re-treatment work, it is advantageous to use stone

of a larger size, in fact the writer's best results have been obtained with a size commercially known as "$\frac{3}{4}$-in." Mr. Fulweiler.

The writer agrees with Mr. Farrington regarding the excellent results that may be obtained in patching with bituminous materials which are prepared so that they may be applied without heating, as this removes one of the serious difficulties in making a large number of small patches. Furthermore, such materials may be applied at almost any season of the year, and are much less affected by the presence of dust and moisture in the road surface.

In this day of definition and striving to attain accuracy in the use of technical phrases, the writer would like to inquire whether Mr. Farrington believes it to be good form to use a proprietary trade name in the way he has used the terms "Tarviating" and "Re-tarviating"?

In reference to the use of tar in re-treatments, the writer's experience has been that where the road is in good condition and an initial application of $\frac{1}{2}$ gal. per sq. yd. is made, that under favorable conditions, patching is all that is necessary during the second season, and about $\frac{1}{3}$ gal. per sq. yd. for the third season, followed by $\frac{1}{4}$ gal.; although, under many conditions, $\frac{1}{3}$ gal. every other season seems to be sufficient.

The writer does not agree with Mr. Farrington in reference to applying the covering to the bituminous material lightly at first. After a little experience it is very easy to estimate the proper quantity, and better results are obtained by putting on this correct quantity immediately. This saves trouble, annoyance to traffic, and prevents the formation of spots in the road, where, owing to the small quantity of covering, the road surface will pick up, especially under heavy steel-tired traffic. It is but fair to state that the writer is drawing purely on his own experience, which has been confined practically to a single type of material.

Slipperiness frequently results from the method of putting on the stone covering, as suggested by Mr. Farrington, as the application in small quantities tends to keep an excessive quantity of bitumen in the top of the road.

In connection with the description of the organization for surface treatment and re-treatment, the writer would be very glad to have an expression of opinion from Mr. Farrington as to the number of square yards per day treated by such organizations, assuming an average haul of 3 miles from the railroad station.

Mr. Farrington mentions a macadam binder, or the heaviest road oil to be used in re-shaping the surface of an old road. By macadam binder, does he mean a particular grade of bituminous material, or the usual stone screenings well watered and rolled?

In reference to maintenance work on screened gravel, the writer is in entire agreement with Mr. Farrington, that a fairly light grade

Mr. Fulweiler.

of bituminous material should be used on the first application, followed by a heavier grade, but finds it best to apply the heavier as soon as the light material has been absorbed into the surface. This same method has been used successfully on oyster-shell roads.

In re-surfacing old bituminous pavements, the writer's instinct would always be to advise scarifying, but the results of some work done during the past season would seem to indicate that, with care, it is possible to secure very good results by using crusher-run stone, free from dust, that will pass a 1½-in. screen, applying this as a covering over about ½ gal. of material, rolling thoroughly, applying about ⅓ gal. and covering with stone chips.

In the organization for re-surfacing, where a considerable amount of this work is to be done, the writer would like to call attention to a plan by which a very large amount of such work has been done on the State roads in Maryland during the past several years. Under this plan the State furnishes and applies the necessary stone covering, purchasing the stone and having it delivered in piles along the road-side in the early spring, when the price of teams is quite low. From a schedule prepared by the State engineers, showing the location of the roads to be treated, a price is agreed on, and a contract drawn up whereby the manufacturer of the material agrees, for a unit price per square yard, to sweep surfaces which have been previously treated with bituminous material, and also surfaces which have not been thus treated, and a unit price per gallon for applying the material, at the approximate rate shown on the schedule. In this contract for the past season, it was specified that all work should be completed before July 31st, with a penalty clause of $25 per day for every day that this time was exceeded. There was a further clause in which the company agreed to pay the State $25 per day for every day that it was delayed through breakdowns or the failure of material to arrive, and the State agreed to pay the company the same amount for every day that it delayed the application of the material through inability to cover it. This contract involved some thirty-four pieces of road in nine counties of the eastern shore of Maryland, amounting to about 118 miles, and containing 976 071 sq. yd. On this was applied 342 785 gal. of material, at a rate varying from ⅓ to ½ gal. per sq. yd., and averaging 0.366 gal. per sq. yd. This was covered with stone chips averaging 12 lb. per sq. yd.

Two motor trucks were used in this work, and they arrived about May 26th, although, owing to bad weather and some delay in organizing the gangs, work was not commenced until May 28th. One truck finished practically half of the work on July 18th, and the other finished on July 30th. The State maintained two gangs, of 12 men each, each under an inspector, for applying the stone chips, the company sweeping the roads and furnishing and applying the binder.

Mr. Fulweiler.

Practically this same form of contract had been in force during 1912, but during that year the work was not laid out according to schedule, and the trucks frequently had to make long jumps between the different pieces of work. This caused considerable loss of time, which was furthermore augmented by difficulty in securing proper labor. As the result of the experience of 1912, the work was laid out more systematically in 1913, and men composing the covering gang in one case were held throughout the season. The effect of having experienced men for covering is well brought out in Table 1 which summarizes the operations of the two years.

TABLE 1.—SUMMARY OF OPERATIONS FOR 1912 AND 1913.

	1912.	1913.
Sunday and holidays	21	20
Repairs and delays	22	6
Rain	5	9
Transit	34	18
Applying material	57	72
Total elapsed days	138	125
Percentage of days worked	66%	72%
Percentage of days material applied	41.5%	57.5%
Average No. of gallons applied per day	2 860	4 760
Average No. of square yards treated per day	8 230	13 550

The time is given in truck-days.

In 1913 the decrease in time lost in transit, repairs, and delays, is quite striking, as is the average increase in the quantity applied per day.

The writer believes that, in the long run, this plan, where there is a considerable mileage to be treated, is probably the most efficient and economical method of handling this maintenance work. One point, which is of considerable importance, is that, contrary to the case of an ordinary contractor, the company is vitally interested in securing the very best possible results from the use of its material, and therefore the inspector's duties are very much lightened, as there is no incentive to skimp the sweeping or other details in the application and then blame the quality of the material for the poor results which would inevitably ensue.

Mr. Sturdevant.

JAMES H. STURDEVANT, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Farrington has well covered the subject of maintenance, without which any road system becomes an abomination instead of an improvement. Many times the writer has listened to tourists recounting a day's experience on the roads. They have expressed great indignation concerning some rough spots, but have not mentioned the many miles of smooth, easy-riding roads over which they have passed.

There are a few minor points to which the writer would like to call attention. These may seem to be trivial, but, if attended to, will result in ultimate satisfaction to the general public.

Mr. Sturdevant.

Patching.—In order to keep the surface of a road in smooth, proper condition, it should be under constant surveillance, and receive constant attention. As the surface becomes worn in places, and holes and ruts develop, they should be repaired at once, for if neglected they rapidly extend in size and depth under the action of each traveling vehicle. On bituminous surfaces, the best results are obtained, as stated by Mr. Farrington, if the asphaltic materials are first mixed with stone, gravel, or other suitable ingredients. This mixing can be done at any time, and in sufficient quantities to last for several months, for the weather will not cause the mixed material to deteriorate. The writer has used material which had been mixed during the previous summer and piled by the roadside, and has obtained as good results as with fresh materials.

In all patching work, the first requisite is to see that the surface to be repaired is well cleaned before new material is placed; also, in repairing holes and ruts, care must be taken not to use too much asphaltic material, and it is well not to have the new patch flush with the surface, for, under the action of the sun, the asphalt, whether light or heavy, will come to the surface. Then the patch will become sticky and is likely to be pulled off by heavy slow-going vehicles. To prevent this, more stone and fine material must be used as covering, and, if too much was used originally, a hump will develop which is as disagreeable as the original rut or hole. This is not as likely to occur when the material has been mixed before placing, as when the asphalt is placed first and then covered. Where the patching has been properly done—judging from the writer's experience and observation—it will last long after the original oil and surface have disappeared. This is true as to either light or heavy oils, or even asphaltic binders.

Re-oiling.—For re-treating bituminous pavements, the quantity of material to be used varies with individual roads. The original treatment should be in sufficient quantities to bind the road thoroughly and hold the stone in place, to prevent raveling; subsequent applications are for the purpose of providing a mat or cushion. For this purpose, the lighter oils are generally used, though excellent results, well worth the extra cost, are obtained with heavy binders. Considering, however, the lighter grades of oil, it is apparent that if used in excess they are not heavy enough to sustain the weight of traffic, and a rutted condition is sure to result. To avoid this, the application of oil should be uniform and of just sufficient depth to cover the surface, with no free oil remaining. After being covered, the surface should be watched closely, especially in warm weather, and as soon as the oil appears through the surface, more material should be added. In general with all bituminous surfaces, satisfactory results can only be obtained by keeping close watch of the road and repairing all defects as rapidly as they appear.

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THE EFFECT OF SATURATION ON THE STRENGTH OF CONCRETE.

Discussion.*

By J. L. Van Ornum, M. Am. Soc. C. E.†

J. L. Van Ornum, M. Am. Soc. C. E. (by letter).—It is gratifying to note the several instances cited as showing the marked change in strength which results from saturating concrete which has been previously exposed in air. Undoubtedly there are many others, that have not been reported, which would indicate the same general tendency. Of course, the proportionate loss of strength occurring when an air-cured concrete is saturated would depend on a multiplicity of existing conditions, some of which have been referred to. One of these is the relatively large area of surface compared to volume in the test specimens, as noted by Mr. Worcester. It would seem probable, however, that the difference in loss of strength between the concrete in structures and that of the experiments would not be nearly as great as suggested by those comparative differences, because there is the partly compensating fact that structural members are generally exposed during construction to the drying effect of breezes and often to direct sunlight and a dry air, while the specimens were in a small closed room in which the air was comparatively moist. If, on the contrary, "the surface were kept wet for 7 days," it is believed that a large part of the loss on saturation would be avoided. Mr. Van Ornum.

With regard to the values of deformations and sets, which Professor Hatt assumes were withheld, it can only be stated that circumstances prevented their determination in connection with the tests reported. The only additional information pertinent to the subject, which can

* Continued from January, 1914, *Proceedings*.

† Author's closure.

Mr. Van Ornum.

be added, has reference to the suggestion of Mr. Worcester, that the actual strength of the specimens cured entirely in air "must have been rather lower than would naturally be expected of this quality of concrete." These cylindrical prisms averaged 1760 lb. per sq. in. Though this is about one-tenth less than the compressive strength that may be fairly expected of good laboratory specimens of these materials and proportions, they were actually somewhat stronger than the average of a large number of test prisms of similar concrete used in building construction and supposedly treated during curing in a way corresponding to that given to the material in the structures.

The writer hesitated to publish the paper at all, because of the many supplementary questions remaining for investigation, the determination of which is necessary for a complete understanding of the phenomena involved. Nevertheless, he concluded that the field of further research would be made more definite and available to others, if he presented the preliminary results already given. Of course, other percentage values will result from different experimental details, and the curve itself may be varied considerably in the same way. Yet it is believed that the broad, general fact of a significant loss in the strength of dry concrete when suddenly saturated and its gradual increase in strength to a final value considerably greater than the original one, when the saturation is prolonged, is a reality which should be definitely recognized in dealing with this important engineering material.

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MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

By G. E. P. Smith, M. Am. Soc. C. E.

Mr. Smith.

G. E. P. Smith, M. Am. Soc. C. E. (by letter).—The author's discussion of weirs is particularly timely, inasmuch as the necessity for the measurement of water by all water users is being felt at the present time. There are comparatively few irrigation projects in the Southwest where the water is not now delivered, nominally at least, by measure.

The author has rendered valuable service in collating the various masses of experimental data on the flow over suppressed weirs. There appears to be a general agreement among experimenters within 3%, though a divergence of results as high as 5% is not infrequent. Apparently, there is need of another set of experiments in which conditions and methods are defined much more rigorously than any thus far attempted. If an important new set is made in the future, doubtless the author will wish to amend his formulas. On account of the nature of the problem, no formulas can be accepted at the present time as final.

The Francis formula, a sort of average law, has been disintegrated into a multitude of formulas, each covering a limited range of conditions. This is in the interests of accuracy, and here the author has succeeded very well, but the writer cannot agree with him that it is in the interests of simplicity. Of course, the general use of accurately platted diagrams avoids the use of cumbersome tables, but errors in taking results from the diagrams can easily exceed the discrepancy between the author's and the Francis formulas.

* Continued from January, 1914, *Proceedings*.

Mr. Smith.

One assumption made at the outset of the paper is perhaps open to question. It is that the discharge is proportional to the length of crest. The influence of the side-walls on the discharge must be variable, depending on the material of which the walls are built and the character of the workmanship. The influence is comparatively slight for long crests, but of considerable importance in the case of short ones, such as are found in irrigation laterals and head-ditches. A serious difficulty in applying the author's method is the standardization of the side-walls. It is much simpler to standardize an edge than a wall.

It would seem to be unnecessary, too, to place the gauge so far up stream as is recommended. The object being to get back of the "surface curve," which depends mainly on the head, it is more reasonable to place the gauge at a distance back dependent on the head under ordinary or average conditions of the stream, say, at a distance equal to four times the head.

The writer formerly used the standard rectangular weir, but abandoned it gradually in favor of the Cippoletti weir, on account of the greater ease and simplicity of installing the latter. The Cippoletti weir requires much less construction than a suppressed weir, a pool being formed up stream in place of a flume. On one occasion the writer built and set an 18-in. weir, got a satisfactory reading and caught a train in 80 min. The Cippoletti weir discharge is proportional to the length, and a table of discharges is so simple that an engineer is not required to interpret it. Probably the greatest error in its use is in neglecting the velocity of approach, and this error can be minimized by providing an ample height of weir, and by applying the usual Francis correction. Furthermore, it would be possible to analyze the Cippoletti weir in a manner similar to the author's, and the subsequent use of the "approved" tables would result in the highest degree of accuracy possible with weirs.

Whatever doubt the writer had regarding the accuracy of the Cippoletti weir was set at rest by observing an exceedingly valuable set of experiments in the hydraulic laboratory of the University of Wisconsin. These tests were performed by senior students under the direction of Professor G. J. Davis. They indicated that:

1. The ratio, 1:4, is correct for the side slopes;
2. The coefficient, 3.37, is correct for crests more than 6 in. long;
3. The error of the formula will be less than 1%, if the head does not exceed 0.4 of the length of crest.

The restriction sometimes placed on the rectangular weir is that the head shall not exceed one-third the length of crest.

Mr. Smith.

There is one idea expressed in the paper to which exception should be taken. The proposition to establish one type of weir for universal use by legislation is debatable at least. Legislators are not engineers, and the fewer legal restraints thrown around the Profession the better it will be for the Profession. There are no laws fixing the power of the transit telescope, or the size of sewer pipe, or steel rail sections, nor are such laws wanted. The standard unit of measure should be fixed, as it is, but the method, never. Better methods will be found in the future, and should not be debarred from use in advance. There are several good ways to measure land, to measure electric current, and to measure water. The best method for a particular case depends on the surrounding conditions.

Utah has a law establishing standard cross-sections for the roads of the State. Is it possible that the southern counties and the northern counties require the same road crowning and the same shape of gutters? In Arizona roads are built variously of volcanic cinders, gravel, out-wash deposits, caliche, and sand; some roads are used almost wholly by automobiles, others by burros; some are in pine forests, others where the rainfall averages less than 3 in. per year; some withstand traffic principally, others rain and frost, others terrific winds. Surely no standard cross-sections are applicable here. Logan Waller Page, M. Am. Soc. C. E., states that "individual judgment is necessary" to determine the cross-sections of roads.

Instead of more standardization, perhaps more originality is needed. Engineers sometimes follow the well-beaten track when much could be gained by taking the cut-off—a statement which is worthy of the following illustration. For several years the writer had charge of a weir in a mountain canyon near Tucson. The discharge of the canyon varies from a small trickle in the dry months of the drier years to occasional floods of more than 1 000 sec-ft. It was quite as essential, perhaps more so, to determine the discharge of the years of minimum run-off as that of the infrequent maximum years. What was needed then, was an adjustable length of crest. Another consideration was the shape of the canyon; there were vertical rocks on the north side, at the foot of which ran the small flows, and a rising bouldery streambed from there to the south side. The weir crest, 65 ft. long, was designed to fit the canyon. For 5 ft. on the north side it was level, and the remaining 60 ft. had a slope upward of 1 in 15. Consequently, the small flows were concentrated over the short-level portion. The weir was built for one-third of what it would have cost with a crest of standard shape. The discharge formula for the weir is derived by using Francis' formula as a basis, and integrating separately for the triangular portion. The differential equation for the latter is

$$d\ Q = 3.33 \left(\frac{x}{15}\right)^{\frac{3}{2}} dx$$

Mr. Smith.

and the discharge for the entire cross-section is given by the equation,

$$Q = 16\tfrac{2}{3}\ H^{\frac{3}{2}} + 20\ H^{\frac{5}{2}}.$$

The equation was tabluated and also platted as a curve. Generally, the curve was used in working up the nilometer records.

There were errors, of course, due to neglect of the velocity of approach, and, probably, to submergence during extremely large floods. The influence of the former could have been approximated, but it was desired to have any possible error on the safe side. The action of the flow was such as to keep the up-stream side scoured out to a depth of from 1 to 2 ft. Records were kept for 9 years, during which time the nilometer box was submerged three times. The credit for the design of the weir is due to Professor S. M. Woodward, of the University of Iowa.

The writer, while traveling in the irrigated portion of Italy, saw many different methods of measuring water, among them various forms of weirs, submerged weirs, submerged orifices, and partly submerged orifices. It was interesting to observe that, in most cases, the method used was best fitted to the conditions of fall, discharge, accuracy desirable, and relative cost.

In the western part of the United States it is likely that a modification of the Venturi water meter will come into use to a considerable extent.

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STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

By F. B. Marsh, Assoc. M. Am. Soc. C. E.

F. B. Marsh, Assoc. M. Am. Soc. C. E. (by letter).—This paper bids fair to be the starting point for much future consideration of this hitherto intangible question of the proper storage to be provided in municipal impounding reservoirs. For this reason the writer wishes to call attention to a factor which, although of relatively minor importance, yet becomes worthy of consideration when a water-shed has been developed well toward the economic limit, as, for example, the Croton. Mr. Marsh.

In allowing for evaporation in a well-developed water-shed, the area of water surface should not strictly be taken as that with all the storage full, because such a condition obtains generally for only a few weeks during the year, and in many years not at all. Moreover, the tendency is for it to occur at a season when the evaporation from water surfaces is small. The maximum evaporation occurs during the summer, when, as a rule, the storage has been depleted and the water surface is, therefore, smaller.

Opinions will probably differ as to what should be taken as an average condition of the reservoirs, but for present Croton conditions, at least, it is probable that the water surface corresponding to a condition of reservoirs two-thirds full would be a fair assumption. Certainly it should not be less than one-half.

The fact that the area of water surface is a continually variable quantity has led the writer to prefer stating catchment areas in terms

*Continued from January, 1914, *Proceedings*.

Mr. Marsh. of the total area, land plus water, rather than land area only, as used by the author. The correction for evaporation can apparently be made as readily on one basis as the other, and it has always seemed to the writer more convenient and logical to use the gross area, and less likely to confuse those who may have occasion to make practical use of the figures.

The author's warning that actual records of an individual stream, taken without reference to other data, are likely to be seriously misleading, even if they cover a considerable period, is strongly exemplified by the Croton record, where the average flow for the 18 years from 1869 to 1886 was 346 000 000 gal. daily, while for the next 18 years, 1887 to 1904, it was 449 000 000 gal. daily.

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PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.

Discussion.*

BY MESSRS. H. A. GARDNER, A. W. CARPENTER, MAXIMILIAN TOCH, R. D. COOMBS, LEWIS D. RIGHTS, AND W. E. BELCHER.

Mr. Gardner.

H. A. GARDNER,† ESQ.—In referring to the behavior of various pigment paints, Mr. Sabin states that "our knowledge of paints is as yet largely empirical." This may be true in so far as it concerns any attempt to promote the use of a lead pigment, which is highly unsuited for the protection of structural steel, but it is a matter of great pride to the paint industry of the United States to point out the developments in paint technology which have been recorded during the past 10 years. In probably no other industry have such rapid strides been made. The achievements of Committee D-1 of the American Society for Testing Materials are especially noteworthy. The studies and researches of its members are largely responsible for the present state of this industry, in which the manufacturing processes are now controlled by the application of scientific principles. This change has led to a general abandonment of the slap-dash, secret-process methods of mixing paint, which at one time existed.

Regardless of the progress that has been brought about through research and practical test, it is not uncommon to hear the phrase, "some thirty years ago", repeatedly used by the skeptic in referring to the old-time methods and materials of painting. To assert that there is but one paint or pigment that can be used for painting any and all kinds of structures, is like asserting that one brand of patent medicine should be used for the cure of any and all ills. In these modern days, it is the practice to design paints so that they will

* Continued from January, 1914, *Proceedings*.
† Assistant Director, The Inst. of Industrial Research, Washington, D. C.

Mr. Gardner.

properly meet special requirements. In no other way can the best results be obtained.

Unfortunately, Mr. Sabin's statements regarding the Havre de Grace Bridge tests are incorrect, and therefore misleading. In referring to the excellent results obtained with red lead paints, he states that the red lead used was 98% pure. The analyses made by the Department of Agriculture, at Washington, show that the red lead paints which gave good results varied widely in their composition. No. 6 paint consisted of 65% of red lead, the remainder of the pigment being made up of silica and other pigments. No. 10 contained 88% of true red lead and 10% of litharge. No. 11 contained approximately 95% of true red lead, the remainder consisting of litharge. Moreover, these three paints (the prime coats) were all made with different grades of oil and in different quantities. The oil in No. 6 which was present to the extent of 45%, had an ash content of 10.6, an iodine number of 140, and an acid value of 30. The oil in No. 10 was present to the extent of 30%, had an ash of 10, an iodine value of 157, and an acid value of 22. The oil in No. 11 was present to the extent of 23%, had an ash of 2.5, an iodine value of 163, and an acid number of 14. Paints 12, 14, and 16, which also gave excellent results in these tests when used at the normal spreading rate of 600 sq. ft. per gal., were composed of mixtures of red lead with such pigments as zinc oxide, China clay, gypsum, silica, asbestine, and carbon black.

It is hard to understand how any one can draw conclusions, from such a test, as to the protective value of different single pigments. The Havre de Grace Bridge tests have served a most excellent purpose in determining the relative value of various commercial brands of metal paints. These tests, however, cannot be cited as having contributed any definite knowledge relating to the effect of single-pigment paints as accelerators or preventives of corrosion. In fact, these tests were not designed for that purpose.

The only authoritative paint tests, to determine the efficiency of single pigments as preservers of iron and steel, which have ever been made, are those by the American Society for Testing Materials, at Atlantic City, N. J., in 1908. In these tests, which were made on a very extensive scale, the so-called rust inhibitive pigments proved the most satisfactory and received the highest ratings at every official inspection. Such pigments were either of the basic or the chromate type. The slightly soluble chromate pigments, such as zinc chromate, as Cushman had predicted, gave marked protection to the metal plates. The highly basic pigments, such as basic sulphate of lead, also gave excellent protection to the iron plates, to which they were applied as oil paints. Some of the most startling results of the test were recorded on test panels Nos. 34 and 36, and Nos. 9 and 10. Test panels No. 36 were coated with neutral chromate of lead. This pigment

Mr. Gardner.

is neutral and highly insoluble. It gave, therefore, only fair results. Test panels No. 34 were painted with basic chromate of lead. On account of the basic nature of this pigment (high litharge content), it gave marked protection to the metal sheets to which it was applied; in fact, this pigment was given a rating of practically 100% by every inspector at every annual inspection. At the end of 5 years' exposure to the sea air, this paint is still in perfect condition, and will probably give excellent protection to the iron plates for many years to come. Panels 9 were painted with that type of neutral red lead which Mr. Sabin claims to be a good protective paint. This neutral red lead paint at the end of 3 years' test gave very poor results on every plate to which it was applied. The surface whitened, chalked, and became covered with little rust spots. The neutral and inert nature of this red lead made it of no more value as a metal paint than ordinary clay. On plates No. 10 there was applied a red lead containing a considerable percentage of litharge. At the official inspection of the tests, this basic litharge red lead received a very high rating, and completely demonstrated its superiority over the neutral red leads.

If the engineer is to adopt red lead as a primer for structural steel, he should specify that the red lead used, if in dry form, should be mixed on the job in oil, or if in prepared mixed form, should be a red lead of inhibitive properties. In the speaker's opinion, it should contain not less than 80 or more than 90% of Pb_3O_4, the remainder being litharge. Most high-grade red lead will run between 82 and 90% of Pb_3O_4. The red lead used should be extremely fine; 99% should pass through a 200-mesh screen. It should contain not more than 0.5% of impurities of any type. Engineers who are responsible for the upkeep of steel bridges, railroad equipment, and similar structures on which red lead is used, are rapidly becoming convinced of the value of basic inhibitive red lead. Specifications issued by the Union Pacific and Southern Pacific Railroads, and the City of Pittsburgh, call for a pure red lead having a minimum of 85% of Pb_3O_4. Specifications of the Rock Island System, the Illinois Central Railroad, and the Yazoo and Mississippi Valley Railroad call for a minimum of 80% of Pb_3O_4. The Public Service Commission of the State of New York has also issued specifications for a red lead having a minimum of 85% of true red lead. This red lead will be mixed with iron oxide, silica, asbestine, and similar pigments, and made into prepared paints. These paints will be used on the new elevated and subway roads.

That thoroughly inhibitive pigments are necessary for the protection of steel cars is evident from the practical tests made at the Pullman shops in Chicago, where thousands of steel cars are produced annually. The following quotation from a paper on this subject, as prepared by the chief chemists of the Pullman Company, presented at a recent meeting of the Master Car and Locomotive Painters' As-

Mr. Gardner.

sociation is quite illuminating, in view of Mr. Sabin's statements regarding rust inhibitive paints:

"The pigment must be of such material that it will be inhibitive, or, in technical words, must be alkaline to a slight extent and positive electrolytically to the steel or preferably a combination of both those factors. The Pullman Company, through their laboratory, devised a very comprehensive and careful series of tests, taking into account all the above factors and many more of slightly less importance. All these tests have checked up remarkably well with outdoor exposure tests on steel panels."

As this reference indicates, rust inhibitive pigments of the basic type are being used widely for the protection of metal surfaces. For this purpose, there are many pigments which have proved quite as suitable as the best grade of red lead. Among these may be mentioned basic sulphate of lead (blue lead), zinc oxide, and the red and black forms of iron oxide. Prepared metal paints made from such pigments have proved highly efficient in test and in practice.

In summing up this discussion, the speaker wishes to impress on the engineer the necessity of using, as a priming paint for metal, one which will contain thoroughly inhibitive pigments, either of the basic or chromate type. Manufacturers of lead products may find that they can economically work up their off-color, low-grade, white lead for producing neutral red lead of 98% tetroxide content. Such a product, may possibly be used in the manufacture of rubber goods, glass, or pottery, but is entirely unsuited for application to steel. To recommend such an inert pigment for the protection of iron, shows either a total ignorance of the well-recognized principles of metal protection, or an absolute disregard for the principles of conservation. In the speaker's opinion, it is quite as criminal to recommend a worthless paint for use on the steelwork of a modern structure, as it would be to allow the use of structural steel which is physically imperfect. In either case the ultimate result would probably be the same.

Mr. Carpenter.

A. W. CARPENTER, M. AM. SOC. C. E.—In his brief and well-written paper, Mr. Sabin has given a great deal of very interesting information, from his point of view, on some of the newer theories and practices regarding oil paints for the protection of ferrous metals. He has used the opportunity to deprecate certain theories he does not himself accept, and devotes a large part of the paper to red lead, the paint pigment in which he is especially interested. He states that "when a new theory is offered, some of them [referring to paint chemists] make a great rejoicing over it without first finding out whether or not it agrees with the facts."

In spite of the speaker's sincere respect for Mr. Sabin's knowledge and experience, he is tempted to suggest that he is indulging in a little rejoicing himself over this new red lead theory, without offering much

Mr. Carpenter.

proof as to agreement with the facts. He states (1) that it is now known that the value of red lead depends on the quantity of true red lead it contains; (2) that the finer the red lead, the better it is; and (3) he suggests that red lead with more than 6% of litharge, when mixed with oil, will set in a day or so. This is evidently offered in opposition to another new theory which has been advanced, which is, that a certain proportion of litharge, at least in the form existing in the red lead as a result of the process of manufacture, is a benefit to the material as a paint pigment. The only evidence Mr. Sabin offers in support of his claims is the reference to the tests on the Havre de Grace Bridge, and this is not very satisfactory, as he does not state the litharge content of the two red leads used straight, and there might be some question as to the influence of the pulverized silicate on the third red lead mentioned. On the other hand, the only evidence offered in support of the theory that litharge is a valuable element in red lead, as far as the speaker knows, is the Atlantic City tests, in which ferrous plates coated with paint made with a red lead containing a proportion (stated to be about 12%) of litharge, were better protected after 5 years' exposure than similar plates coated with paint made with orange mineral, which contained practically no litharge.

The speaker's principal object in discussing this paper is to suggest that perhaps both these contradictory theories are wrong, or, at least, not worth the fuss that is being made over them. This suggestion is the result of a series of tests of samples of red lead of varying composition, as shown in Table 1.

TABLE 1.—TESTS OF RED LEAD FOR NEW YORK CENTRAL LINES BRIDGE ENGINEERS' COMMITTEE.

No. of sample.	Chemical Analysis.			Fineness.		Spreading Capacity of Paint.					
						24 lb. lead to 1 gal. oil.			30 lb. lead to 1 gal. oil.		
	True red lead. Percentage.	Litharge. Percentage.	Other elements, approximate percentage.	Relative fineness.	Weight per cubic inch, in grammes.	Square feet covered.	Relative percentage.	Working qualities.	Square feet covered.	Relative percentage.	Working qualities.
1	94.2	5.7	0.2	2	27	23	74	O. K.	30	79	O. K.
2	92.9	7.0	0.01	3	33.6	20	65	Covered poorly and brushed out hard.	29.5	77	O. K.
3	87.7	11.7	0.5	1	23	28	90	O. K.	33.5	88	O. K.
4	84.5	15.2	0.3	3	33.6	31	100	Mixed thin and sagged.	38	100	O. K.
5	80.6	19.1	0.3	1	23	20	65	Brushed out well but ran.	27	71	O. K.

Quantities for spreading tests: ¼ pint of oil to 12 oz. and 15 oz. of lead, respectively. Spreading tests made May 18th and 22d, 1911.

Mr. Carpenter.

This table shows the results of the examination and tests of five samples of red lead, each furnished by a different manufacturer. It shows the fineness of division, the chemical composition, and the spreading capacity when mixed with linseed oil in the usual paint proportions. It will be noted that the true red lead content ranges from 80.6 to 94.2% and that of litharge from 5.7 to 19.1 per cent. The fineness determination was made by the weight method, and is merely a comparative one based on the theory that the fineness of division of materials of equal specific gravity and similar structure of particles will vary inversely as the weight of equal volumes of the materials when equally packed. The weights given were determined by sifting the leads through a 20-mesh wire screen, placed at a fixed height above the measure of volume until the latter was filled, striking the material off flush with the top of the measure, and then weighing the contents. The lead with the lowest true red lead content (Sample 5) was found to be a tie with one other as the finest in division of all five leads; and the lead having next to the lowest true red lead content (Sample 4) was a tie with one other, the one with next to the highest true red lead content (Sample 2), as the coarsest. The other cases exhibit a similar lack of law of relation between true red lead content and fineness of division.

The paint mixtures for the spreading tests were made with open-kettle boiled linseed oil and no drier. They were applied to clean, galvanized, sheet-metal surfaces—well weathered and apparently all in the same condition—by one operator, on the same day, and under the same weather conditions, using a separate clean brush of one size and kind for each paint. The quantity of paint was small, but covered an appreciable area in each case, and the results with the two proportions of lead and oil will be noted as generally consistent. They show that one of the two coarsest leads (Sample 4), and the one having next to the lowest true red lead content, gave the greatest spreading; and one of the finest leads, which had the lowest true red lead content (Sample 5), gave the smallest spreading.

It seems safe to say that, if these tests are a correct indication, there is no definite relation between the fineness, the proportion of true red lead, and the spreading capacity of the pigment in oil, but that other elements control.

The foregoing determinations, of course, do not give any indication as to the durability of the respective leads in paint, nor their respective values for the protection of ferrous metals from corrosion. In the endeavor to determine these qualities, the leads were mixed with boiled linseed oil in the proportions of 20, 25, and 30 lb. of lead to 1 gal. of oil, and the resulting paint was applied, under three different conditions, in one coat each on steel plates 5 by 7 in. in

lateral dimensions. The paint was prepared by first pouring a small proportion of the oil over the full quantity of red lead and letting it stand over night. Next morning, the remainder of the required quantity of oil was added, and the whole was stirred thoroughly. Mr. Carpenter.

Three lots of paint, one for each proportion of 20, 25, and 30 lb. per gal. of oil, were thus mixed from each of the five samples of red lead, thus making fifteen lots. From each of these fifteen lots a steel plate was coated and properly marked for identification. Then from each lot, two small wide-mouthed bottles were partly filled. One of each of these bottles was allowed to stand open, without stirring, for a period of 2 weeks, at the end of which time the paint was stirred and applied to a steel plate in a single coat; in no case had the mixture set so that it could not be readily stirred and applied in a good coat, although in one or two cases the coats were quite thick. The paint in the second set of bottles was stirred every day for 2 weeks and was then applied to steel plates, except that one lot had set so that it could not be applied; that one was made from Sample 5 in the proportion of 30 lb. of lead to 1 gal. of oil; this sample contained the greatest proportion of litharge, 19.1 per cent.

After coating these plates, they were exposed to the atmosphere on the roof of an office building in New York City and left for 463 days, or 1 year and some 3 months. They were then brought indoors and examined with the following result:

The plates to which paint was applied immediately after preparation were in generally good condition. Those coated with 25 lb. to 1 gal., were in the best condition, although the coatings had changed color. There was no choice between the coatings with leads made by the different manufacturers. The 30-lb. per gallon coatings were too thick in some cases, and showed more breaks, with rust spots, than the 25-lb. per gallon coatings. The 20-lb. per gallon coatings showed the best color, probably due to the greater protection of the pigment by the oil.

The coatings with the mixtures which stood 2 weeks before application, in many cases were as good and had protected the steel as well as those applied immediately after preparation; in some cases it was evident that the mixtures had commenced to thicken or "set" at the time of application.

The coatings applied after the mixtures had been stirred daily for 2 weeks gave good protection, with two or three exceptions. They generally were not quite as good as those from the paints which stood 2 weeks without stirring, and the latter were not quite so good as those from the paint applied immediately after preparation. In several cases the paint had begun to thicken up or "set" when applied, but the results from these coatings were decidedly good.

Mr. Carpenter. In view of the foregoing there does not seem to be anything to choose between the different leads, as affecting the durability and protective qualities of paint coatings in which they are incorporated. For economical reasons, the choice would seem to lie with the one that spread the farthest, but there does not seem to be any way to predetermine the spreading capacity of paint made with any particular lot of red lead, from the fineness of division or litharge content of the lead, at least within the range exhibited by the samples tested.

It may be thought that the exposure tests were not long enough for comparative results. In this regard attention is called to the fact that the tests were made on single coats, and that the time was rather long for such.

It is not the intention to offer these tests as in any way conclusive, but simply as some definite determinations, as opposed to mere statements of opinion. It is hoped that others may take up the investigation, to the end that the facts may be established. It would seem to be a good field for investigation by the National Bureau of Standards.

Mr. Sabin's paper would lead one to think that red lead is the only good pigment for paint for preserving ferrous metals. Although it is undoubtedly one of the best of the practical pigments for use under many conditions, there are other conditions under which it is not satisfactory; and it is doubtful if its use in more than one coat is desirable on structural steel for ordinary exposure, owing to its cost, its susceptibility to deterioration in color and integrity in contact with gases, and its often objectionable color.

It may be very well argued that one or two under coatings of red lead paint will be generally improved by over coatings of paints with other and cheaper pigments, rather than additional coats of red lead. The speaker has also known of many instances in which oxides have made remarkable records for durability and protection in paint coatings applied directly to steel surfaces, and, he has a much higher regard for the value of certain iron oxides for use in structural steel paints than the author expresses for this class in general.

Mr. Toch. MAXIMILIAN TOCH, ASSOC. AM. SOC. C. E.—This paper can hardly be said to throw any new light on an old subject; an analysis of all that it contains can be summed up as a propaganda for an alleged new material known as red lead ground in oil, and sold in paste form.

Mr. Sabin's conclusions with reference to the condition of the paints on the Havre de Grace Bridge and the final report are incomplete. The report really describes as excellent nine paints, three of which are red lead; and one of these is a ready-mixed red lead containing, as he says, 15% of a pulverized silicate. Now, to be frank, such an indefinite term as "pulverized silicate" should not be used.

Mr. Toch.

The paint in question contained a substance known as aluminum silicate, but better known to engineers as ordinary American clay. The statement that 15% of the composition of this paint was clay is also misleading. Mr. Sabin, however, has corrected this by stating that the percentage of clay is nearer 25. The specific gravity of clay is a little greater than 2, the specific gravity of red lead is about 9. The volume of these two materials is such that the actual quantity of clay in this paint was nearly as much as that of red lead. In other words, 25 lb. of a good, clean clay has the same bulk as 85 lb. of red lead.

Now, what has the author proved? That the two pure red leads in these three results were only as good as the red lead which was mixed with clay, and, therefore, that clay, when mixed with red lead in moderate proportions, is just as good as red lead, for, after 6 years, there has been no difference in these three test plates. Then again, every one of these red leads was coated over with a carbon paint, and, if the test proves anything, it proves that the carbon paint was a perfect excluder of oxygen and water, and therefore no rust resulted.

The speaker has examined the report of the Havre de Grace Bridge committee very thoroughly, has been active on this committee to some extent, and has inspected these plates. Mr. Gardner has stated that among the paints which have turned out the best was that on Plate No. 12, and as nearly as can be determined from the report, that plate is the only one having two coats of paint, all the others having three coats; and the 600-sq. ft. spreading test is as good as the best. The speaker is not ashamed of this, in view of the fact that it was a paint invented by himself and made by his firm. Bearing out Mr. Gardner's statement with reference to the inhibitive quality of the basic paints, that on Plate No. 12, according to the published analyses, was the only one which was alkaline, due to the fact that the priming coat was a now fairly well known cement paint. The finishing coat was a carbon paint, and after 6 years these two coats are equal to the three coats of the others.

Four smokestacks for the power-house of Long Island City were painted in August, 1906, and are in as good condition to-day as they were 7 years ago. In that case three coats were applied, but the first was painted directly on rust, and that rust has never progressed sufficiently to scale the succeeding coats of paint.

The author has not even "damned by faint praise" any other material as a paint. On December 13th, 1913, the speaker examined very critically the test plates at Atlantic City which were painted in 1908 with single pigments, where, for instance, oxide of iron, red lead, graphite, etc., were applied without any subsequent protective coat, and he must take issue with Mr. Sabin, for the three red lead plates which are still on exhibition there are not as good as the three plates

Mr. Toch. covered with Prince's metallic brown, which is an oxide of iron; nor is the American orange mineral, which is a pure form of red lead, anywhere near as good as the sublimed blue lead. In fact, to be perfectly frank, the sublimed blue lead is far better than either red lead or orange mineral.

Allusion must be made to Mr. Sabin's statement: "one should not be disturbed by talk about 'newer paint oils'." The author did not mean "newer paint oils" when he put those words in quotation marks, but intended to refer to "newer paint materials," which was the subject of a lecture delivered by the speaker in London nearly 3 years ago, and the speaker is quite sure that Mr. Sabin is not so much against progress as this statement would indicate. Considerably more than 100 000 copies of that lecture were reprinted, and it has been quoted extensively both in Germany and in France, therefore, when Mr. Sabin pretends that there are no newer paint materials, the speaker is not inclined to take him seriously.

The speaker has always held the strong opinion that no man should take out of the common store any more than his allotted share. To him it is just as criminal to corner the wheat market and raise the price of bread as to take money out of the pockets of other people; there is this difference, however, the pickpocket can be prosecuted, but the man who raises the price of bread can not.

About 4 years ago, at a time when the crop of turpentine was normal, some men in the South cornered the market. Hundreds of thousands of gallons were stored, and none of it was sold until the price, which should have been about 45 cents, was raised to $1.13 per gal. The speaker very quietly cast about for a substitute for turpentine, and so did many others. There was one man, in the employ of the United States Navy, who showed much more courage than any admiral ever did in battle, when he wrote a specification for turpentine substitute for the United States Navy, and in one year reduced the purchases of turpentine in the Navy about 70 000 gal. A large number of other progressives did the same thing, with the result that to-day we can do without turpentine absolutely, and the price in its tumble shattered those who had raised it abnormally.

About 3 or 4 years ago, through the failure of the flaxseed crop in the United States and other countries, the price of linseed oil rose from 36 cents to $1 per gal. Although the speaker has never contended that he had found an oil which was just as good as linseed, he has proved that there are several which, in conjunction with linseed oil, make most excellent paint. If the author thinks there is nothing in the newer paint materials, let him ask the manufacturers of printing ink, linoleum, artificial leather, and also a number of paint manufacturers, whether Soya bean oil and Menhaden oil are not valuable assets to them to-day. A few months ago one of the largest manufacturers

of Menhaden oil stated that where he sold one barrel to the manufacturers of paint and linoleum 10 years ago, he is selling a thousand barrels to-day. In the speaker's lecture on fish oil he pointed out that many of the failures were caused by the ignorance of the men who made the tests. On any price list of fish oils to-day will be found whale oil, porpoise oil, and seal oil, under the heading of fish oils, and it was the speaker who pointed out to the paint industry that these are animal oils and not fish oils, and, further, that a fish oil, to be suitable for paint material, had to possess certain constants, otherwise it would be useless. Mr. Toch.

In 1896 5 gal. of China wood oil came to the United States. As far as known, it was the only China wood oil imported that year, and the speaker received those 5 gal. During the year ending October 31st, 1913, 5 400 000 gal. of China wood oil were consumed in the United States. This oil, before it was understood, had a very bad reputation as an adulterant and an undesirable paint material. When one talks about adulterants one is treading on very thin ice. Everybody is familiar with the high-priced Epicurean dish known as terrapin. To serve terrapin at $3 per microscopic portion is considered in high society a great feat, and yet, 150 years ago, when a good slave was sold in Maryland, the agreement generally made was that the slave must not be fed on terrapin, for at that time it was considered hardly good enough food for dogs. If China clay, which is the pulverized silicate mentioned by the author, were equal in price to red lead, a feature would be made of its use instead of a secret.

This paper tends to show that there is a new material on the market known as red lead ground in paste form, which can be very easily thinned and made ready for use. Pure red lead, ground in pure linseed oil, in paste form, has been known for many years, and, what is more, the author knows it has been known, because orange mineral, which is a pure form of red lead, has been used by color grinders as far back as the speaker can remember. The speaker made ready-mixed red lead more than 20 years ago, and made it successfully; and it remained in its ready-to-use form. He has no quarrel with red lead, because he thinks it is a very good material. It is not better than sublimed blue lead, nor is it better than certain forms of oxide of iron. It is certainly very much dearer than any other paint, and, on account of its great weight, it is harder to apply than a lighter one; it does not stop progressive oxidation, nor is it a fit material for use on steel which is to be bedded in concrete, not because of the red lead, but because of the oil, which is readily saponified.

Everybody must admit that in some places red lead is a most excellent protective paint, but in other places it does not give such a good account of itself. Then again, every paint chemist knows that there are some red leads that contain nearly 1% of caustic soda, which

Mr. Toch. is quite sufficient to destroy a paint when it comes in contact with water, and all know that the red lead which can be ground in paste form and remain plastic is not the red lead that has been used heretofore, which contains litharge, and the old litharge red lead paint should not now be condemned because it forms a cementitious material.

As a paint under water, red lead is sometimes good and sometimes bad, depending, of course, on whether or not it has been made by the nitrite of soda process.

A large quantity of red lead was used on the steel of the locks of the Panama Canal as a priming paint, three coats being applied in many places. When submerged in the lake water, the red lead turned blue, owing to the action of the sulphur gases on the lead, and the paint was in many places entirely destroyed. Whether this was due to the fact that it contained a small percentage of free caustic soda and thereby became soluble in water, or whether the decaying organic matter destroyed it, amounts to the same thing. Preliminary tests, however, may have demonstrated these defects.

The speaker has at all times stood for fairness and the exact representation of facts with regard to paint materials. He has spoken well of all those materials which his judgment and experience have taught him were good. He has even had the temerity to talk well of his own paints whenever he has had the chance, for the world is large enough to support all, and to give every one a share. He is glad of the opportunity to state that he and Mr. Sabin have been friends for many years, and he respects him highly. In differing in opinion with him, he is simply breaking a lance in the interest of paint science. The author is a man who is well equipped, and has had a long and varied experience which entitles him to the right to talk, but the speaker agrees with the poet Goethe, *"Der Mensch verhoent was er nicht versteht"*, which, translated into good "American", means "A man is down on a thing when he is not up on it".

Mr. Coombs. R. D. Coombs, M. Am. Soc. C. E.—The manner in which paint is ordinarily applied in commercial practice is so irregular and faulty as to render any direct comparison inaccurate. The painting, as usually done on structural steelwork, is about the poorest job on the entire construction. As most erecting engineers know, the work of painting is disliked by the higher priced men—the iron-workers. Instead of having the work done by a $4 or $5 iron-worker, it is done by a $2 to $3 steel painter or apprentice, and it is frequently a very poor sample of painting. Again, in the entire painting, the shop coat would seem to be the thing which can most easily produce a poor result.

The average paint used in the shop is a relatively cheap material, costing about one-half as much as a good field coat, and it is frequently put on by the cheapest labor in the shop, and more frequently it is put on very badly. Now, it would appear reasonable to expect that

Mr. Coombs.

a poor shop coat over an imperfectly cleaned surface would have a very great bearing on the final net result of the work. Thus, if a $0.60 paint is put on in the shop as badly as is frequently the case, and then a $1.25 paint is used in the field, it seems open to question how much of the result of the final painting is chargeable to the shop coat and how much to the field coat.

Referring to the matter of red lead paint as practically applied—not as a laboratory test, but as done in the field—the question arises, how many pounds of red lead to a gallon of oil should one expect low-grade painters to use successfully. There are in existence a number of specifications, issued by individuals and corporations of high standing, calling for 33 lb. of lead to 1 gal. of oil. On a rather important piece of construction, in the vicinity of New York City, the master painter, who was in the employ of the company and had no object whatever in cheapening the work, reported that he could not put on as much, unless he was allowed to use a trowel. This may have been an exaggeration, but it is the speaker's recollection that about 22 lb. was as much as the men found they could work with to advantage.

The speaker does not wish to convey the impression that the painting in the field is a much better piece of work that that done in the shop, but that the painting, either in the shop or in the field, as usually done, is not a good job. Further, the manufacturer alone is not to blame, but also the owner and the engineer. It is simply a bad practice which has become very common, and for which all concerned are at fault. Reference is not made particularly to red lead paint, but to all paints.

There seems to be some tendency on the part of the manufacturer to regard the multitude of paints now specified as a senseless nuisance, and—it must be admitted—with perhaps some justification, from his standpoint. However, having recovered from his chagrin at the receipt of an order embodying another change in paint from the kinds in immediate use in the shop, and after faithfully incorporating the proper instructions in the shop orders, he often takes no further interest in the matter. The paint question is now up to the shop foreman, whose theories thereon are often in need of expurgation as well as elaboration. To the minds of many shopmen, who take a real pride in doing good shop work, any old paint of the specified color is good enough.

Turning now to the paint man, it is difficult to understand how he can escape responsibility for at least part of the skepticism of the manufacturer, engineer, and owner, as many paint salesmen rather permit the impression to get abroad that the paint used by their competitors will eat the steel rather than protect it.

The private owner is not so much to blame, as he is neither an engineer nor a paint expert. Those corporate owners, however, having engineering departments, whose duty it is to conserve their construction interests, can hardly be absolved from blame for careless work.

Mr. Coombs. The speaker is unwilling, although able, to specify definite instances, in which a cheaper grade of paint than that specified has been used, or in which either, or both, the shop and the field painting have been very badly done. Painting in wet or freezing weather, the use of liberal quantities of "thinners" and benzine, painting over scale, rust, cinders, mud, etc., and the omission of paint, are not such rare occurrences as optimists would have us believe.

It would seem that it is about time for a reformation in the methods used in painting. Not so much in relation to the work done under efficient inspection as in the great bulk of work included in small contracts throughout the country, which are either poorly inspected or not at all. Where paint is really needed, it is usually badly needed; and, unfortunately, the work on which the future maintenance will be inadequate is just the work on which the original inspection and superintendence is inadequate.

It is the speaker's opinion that, in a great many cases, particularly for interior steel beams encased in concrete, no field paint whatever is necessary, and that the expense of painting such members would more properly be expended in the better painting of the members which really need it. Further—although it may seem contradictory to the previous remarks—better results might often be obtained by using a $5 man and $1 paint, than a $1 man and $5 paint.

As a result of some years of experience with a class of construction in which there are very thin members subject to an extreme and variable exposure, the importance of a revision in the common attitude toward the paint question appears to be of considerable importance. Such work may be regarded as an accelerated test, and as developing in a few years what may be expected in a longer period in heavier material, under incompetent supervision.

The speaker would like to ask Mr. Rights what he thinks the average price of shop paint is throughout the country, that is, considering the average of the total number of shops, not the average of the total tonnage.

Mr. Rights. Lewis D. Rights, M. Am. Soc. C. E.—The speaker, as a member of the American Society for Testing Materials, has attended the conventions at Atlantic City, has inspected the paint fence, and has listened to discussions regarding the values of inhibitors and stimulators. He has followed Mr. Sabin's paper and the discussions with a great deal of interest, with the hope that something would be brought out to help to clear up the situation, from the standpoint of one of the parties who is as much interested as any of the rest, if not more so. He refers to the bridge and structural manufacturer, who has the responsibility for the application of the paint, both in the shop and in the field, and is therefore a very important link between the paint manufacturer and the engineer and owner.

Mr. Rights.

It may not be out of place to mention the conditions under which painting is done at the shops of the structural and bridge manufacturers. So far as the speaker knows, very few of the bridge shops have any adequate facilities for doing their painting inside a building, where the work can be done regardless of wet or freezing weather. Although it is true that during the most severe winter conditions painting is done occasionally in the finishing end of the shop, facilities there are so cramped that the quantity of painting done under cover may be said to be almost negligible. It is safe to say that practically all the structural steel painting is done in the open.

Some years ago, the owners of one of the largest shops carefully considered plans for covering their shipping yards and protecting them so that painting could be done at all times. The item of expense, however, was such that it was decided that the condition of the business would not warrant increasing the cost of painting to the owners.

The bureaus which inspect a large proportion of the fabricated steelwork have a rule that the finished material must not be painted until their men have had an opportunity to go over it. Frequently, it takes an inspector several days to cover the shops in his vicinity, and this means that the steel must sometimes lie in the open for a day or so before it can be painted. Now, considering that painting cannot be done in either wet or freezing weather, nor can it be done until the work is inspected, it will easily be seen that the time allowed for doing the painting is very much reduced, and therefore, of necessity, the application of the paint at the shop must be a somewhat hurried job. In general, the class of labor doing the shop painting is fully as good as that employed in the other details of manufacture, and considerable care is exercised in endeavoring to see that all parts of the pieces are evenly covered.

In connection with the erection of structural steel, it has become the practice of many of the bridge manufacturers to sublet the painting to men who make a specialty of work of this class. It has been found that bridge erectors do not relish handling a paint brush, and better results have been secured with specially trained men. As the manufacturers are interested in producing a high grade of work, it has become the custom for them to buy the paint and ship it to the job in unbroken packages. It is then sold to the painting sub-contractor at an agreed price per gallon, and the work of applying the paint is inspected either by the owners' inspectors or by some one appointed by the bridge manufacturer.

The speaker would like to state here that the bridge manufacturer is not in favor of cheap paint. The fact of the matter is that he would rather put on a good paint, provided the cost of this good paint has been properly covered by the specification, and figured in his original bid. It will easily be seen that the more work the bridge manu-

Mr. Rights. facturer can put into a job, the greater will be his percentage of profit, and that, in addition, he will produce a more sightly structure. The bridge manufacturer carries at his shop several lines of standard paints, which are probably as good as can be bought under ordinary specifications, such as red lead, white lead, iron oxide, and some form of carbon or graphite. He is entirely willing to apply the best of these, as called for, or to provide a better paint, if required. What he most desires, however, is that the discussions which have been going on among engineers for the last few years will produce something in the way of standard kinds of paint suitable for each class of work. As matters now stand, it would seem that almost every engineer was working out his own theories of inhibition and stimulation, and that each one had a different variety of paint. Of necessity, the bridge manufacturer must order a somewhat larger quantity of paint than is actually required for each particular structure, with the result that the paint shop has the appearance of a medicine shelf, containing left-overs, each one strongly advocated by one individual engineer, and strongly condemned by all the rest of them. These left-overs will not remain fit for use very long, and are frequently a total loss to the manufacturer.

Years ago, a similar condition applied to specifications regarding the quality of structural steel. To-day, the manufacturer and the engineer have been able to get together, so that the same grade of structural steel is used for railway and highway bridges, buildings, and cars, and when the total tonnage is considered, the special-grade steel is a small factor. Cannot the paint manufacturers and the engineers get together on some such broad basis, and settle on paints which will be serviceable to the owner, without undue cost, which can be readily applied by the manufacturer, and which will fit 90% of the steel structures. "'Tis a consummation devoutly to be wished".

Mr. Belcher. W. E. Belcher, M. Am. Soc. C. E. (by letter).—This subject is certain to hold the attention of a large proportion of the members of the Society, for all are looking for more information. The writer, however, confesses to a feeling of disappointment in reading the paper, particularly in regard to the very small number of paint materials selected by the author for consideration, and also because of his failure to cite from his own wide experience definite records of paint performances from which engineers might be assisted in forming their own conclusions.

The writer would like to add the following random notes from his experience:

For several years a red lead paint was always specified by him for structural steelwork for buildings, following the example of many

railroad and Government specifications. When inspecting the work done under practical shop conditions, the paint was found to be more or less unsatisfactory. There was a decided tendency for the formation of a hard sediment at the bottom of the paint bucket, and ordinary paint laborers never stirred the paint with sufficient frequency to obtain a uniform coat; this would upset fine calculation as to the number of pounds of lead per gallon of oil. There are on hand in most bridge shops several kinds of red paint which can hardly be distinguished from one another immediately after application, and might easily be substituted for one another—unintentionally perhaps—without subsequent detection. The writer has seen on detailed shop drawings the paint note, "One Coat Red," though the original specification had been quite elaborate. If specifications could be reduced to this simple form there would be no difficulty in getting what is called for. Another point which seems detrimental to the use of red lead is the fact that its color soon becomes dull after exposure, without regard to the number of coats applied. Mr. Belcher.

The writer has always been prejudiced against graphite paints, on account of their extreme thinness and also because of the very conflicting arguments advanced by the manufacturers of the pure pulverized graphite paint, on one hand, and those of the mixed silica and graphite, on the other. A specification calling for Prince's mineral and for iron oxide to which graphite was added in small quantity for smoothness and color, was tried out with good results, and has since been followed. It overcomes very largely the objections above offered to red lead, and has generally good lasting qualities.

The preparation of a ground red lead paste mixed in oil would not appear to be entirely new, as one of the large paint manufacturers in Chicago submitted to the writer about three years ago a preparation called "Liquid Red Lead." This was a concentrated red lead paste to which, if memory serves correctly, a small quantity of silica was added, which assisted in an effective way against sedimentation.

This material was submitted in connection with a study of the subject of gas-holder paint. One manufacturer, submitting samples at that time, offered "Special Gas-Holder Red" which was recommended to fill every possible requirement, although the description of its manufacture was very vague. Investigation disclosed the fact that the manufacturers were as little certain of the desired ingredients of a gas-holder paint as the writer was, and, consequently, they put in a little of everything, including old house-paints, surplus paint stock, kettle scrapings, paint skins, etc., with enough red to color the mixture.

The writer is a firm believer in the theories of Messrs. Cushman and Gardner in regard to the corrosion of iron and steel; at the same time he must agree fully with Mr. Sabin that it is practically impos-

Mr. Belcher.

sible for an engineer to use them in their present state as a guide for intelligent selection of a paint material. Has any manufacturer or manufacturer's representative ever been forced to admit that his particular combination of oils and pigments has been in any way discredited by these theories and experiments? The author's remarks regarding lampblack illustrate this point. We have learned that lampblack is in itself injurious. The manufacturer then states that he has ground it in a certain way which changes its action entirely.

The science of engineering is largely empirical, based on observations and experiments. The present discussion in regard to painting structural steel emphasizes the incompleteness of our record of experiments in this particular branch of the science. In the present situation, our rules are still in the nature of theories and deductions from limited personal experience.

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STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

BY MESSRS. L. H. NISHKIAN AND WILLIAM CAIN.

Mr. Nishkian.

L. H. NISHKIAN, ASSOC. M. AM. SOC. C. E. (by letter).—On page 2083† the author computes the shear in section *N I*, Fig. 4. An important function of the inclined bar is to relieve the shearing stresses in section *N I*, but the author does not take into consideration the vertical component of the stress in the inclined bar. In order to have equilibrium of all the vertical forces acting on that portion of the base to the right section *N I*, the vertical component of the stress at section *N I* in the inclined bar should be deducted from the total vertical load, 18 333 lb., and the remainder will be the shear in the concrete in section *N I*.

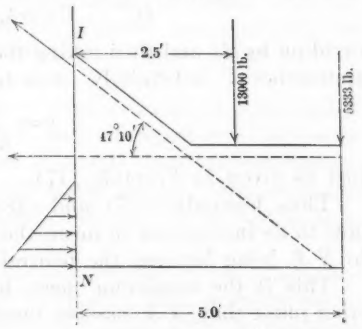

FIG. 12.

The sum of the vertical components, per foot of width of the stresses in the inclined bars, is

$$17\,750 \times 0.765 \times \frac{3}{2} \times \sin.\ 43°\ 10' = 14\,000 \text{ lb.}$$

and

$$18\,333 - 14\,000 = 4\,333 \text{ lb.}$$

Therefore the maximum shear in section *N I* is $47.6 \times \frac{4\,333}{18\,333} = 11.2$ lb. per sq. in. instead of 47.6 lb. per sq. in.

*Continued from January, 1914, *Proceedings*.
†*Proceedings*, Am. Soc. C. E., for November, 1913.

Mr. Cain. WILLIAM CAIN, M. AM. SOC. C. E. (by letter).—The writer would like to make a correction as to the meaning of v in Formulas (17) and (18), as others besides Mr. Janni* may inquire the reason for the statement that "The unit stress, v, acts parallel to $N\ N'''$". The statement is incorrect; v, in Formulas (17) and (18), is the unit shear on any plane, lying between O and the steel bars, perpendicular to $N\ I$.

Thus, suppose such a plane drawn in Fig. 8 and marked $A\ A'$ (A lying in $N\ I$, A' in $N'\ I'$), then since the section, $N\ I$, was taken parallel to the direction of the loads, at a point, A, where the bending stress is supposed to be zero, there is shear only on the plane, $N\ I$, and consequently a shear of equal intensity, v, on a plane, $A\ A'$, at right angles to $I\ N$, by a well-known theorem. The total shear on the plane, $A\ A' = v\ b\ dx$.

For equilibrium, the algebraic sum of the components perpendicular to $I\ N$, of the forces acting on $N\ A\ A'\ N$ must be zero. Therefore

$$(C' - C)\ \cos.\beta = v\ b\ dx,$$

which is the same as the equation just above (17). Hence, substituting this value in the equation derived just before the last, or in,

$$(C' - C)\ \cos.\beta\ D\ G = V'\ dx,$$

dividing by dx and then taking the limit, as dx approaches zero and V' approaches V indefinitely, it is found that,

$$v = \frac{V}{b\ \overline{D\ G}}$$

just as given in Formula (17).

Thus Formulas (17) and (18) are unchanged in form, but v is now to be interpreted to mean the unit shear on a plane perpendicular to $N\ I$, lying between the neutral axis and the steel.

This is the maximum shear, being greater than the shear on any other plane than $A\ A'$ passing through A; hence it is of special interest to the designer.

* In his discussion, *Proceedings*, Am. Soc. C. E., January, 1914, p. 239.

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A STUDY OF ECONOMIC CONDUIT LOCATION.

Discussion.*

By Messrs. H. Hawgood and M. F. Stein.

Mr. Hawgood.

H. Hawgood, M. Am. Soc. C. E. (by letter).—The location of conduits for irrigation and power purposes should receive the same study as that for a railroad. Both are transportation problems, and the economic value of distance and rate of gradient are relatively as important in the one enterprise as in the other. Their importance in railroad transportation is fully recognized.

Mr. Hickok's paper is the outcome of studies made by him, under the writer's instruction, in connection with a Southern California project. His treatment of the subject is simple, and his conclusions are logical. The unit costs used in the paper are not fully endorsed by the writer, but, inasmuch as they are introduced for exemplification purposes only, they are not debatable.

In arriving at methods of comparing the economic values of alternative locations, the fact should not be overlooked that, however satisfactory the mathematical treatment, the conclusions must be tempered by personal judgment of the risks to which the local environments subject the alternative locations. Experience has shown that the risk of interruption of service is far less for tunnels than for hillside conduits, and weight is to be given to this and other working experiences in making a final selection of routes.

Mr. Hickok does not credit tunnels with any saving in seepage. Seepage from a water channel is governed, not only by the nature of the channel lining, but by that of the material on which the lining rests. No water would pass through a sieve set in impervious material. The sides and bottom of a rock tunnel may not be, and rarely are, entirely impervious, yet their opposition to percolation is of suffi-

* This discussion (of the paper by C. E. Hickok, Assoc. M. Am. Soc. C. E., published in December, 1913, *Proceedings*, but not presented at any meeting) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Hawgood.

cient magnitude to aid materially the lining proper, and in most cases to reduce seepage to a negligible quantity.

In his graphical representation of results, the author uses a diagonal line for each combination of the costs of two structures. He gives seven different structures; taken in pairs, twenty-one combinations are possible. Twenty-one diagonals, with their legends, would be found to be confusing and impracticable of application. It might also, and probably would, be found convenient to add other types of structures, which would call for a considerable increase in the number of diagonals.

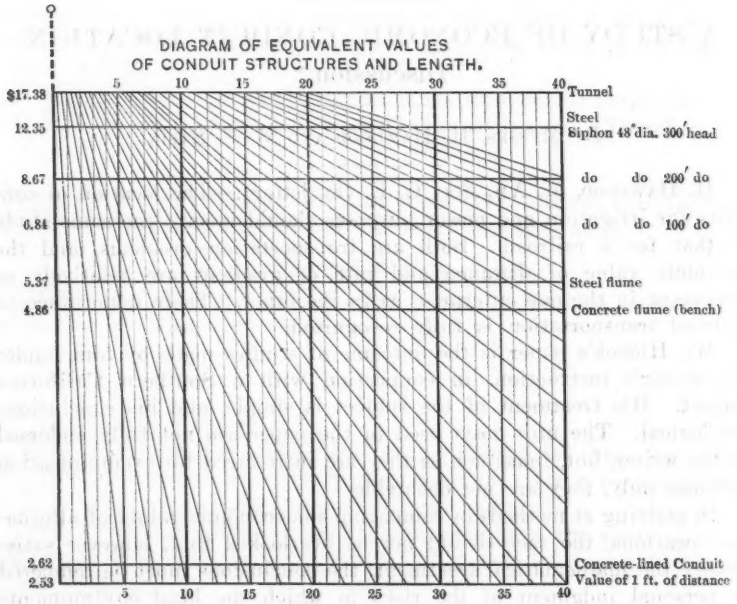

FIG. 3.

The writer suggests Fig. 3 as a more convenient form of diagram and as covering all phases of the subject. The radiating lines permit finding the equivalent distance, or type of structure, in terms of length of any other type of structure. The horizontal lines, each representing a particular type of structure, are spaced so that their distances apart vertically bear the same ratio one to the other as their respective unit costs. To exemplify the use of the diagram, assume that, of two alternative routes, Route No. 1 has 200 ft. more tunnel and Route No. 2 has 1 500 ft. more lined conduit and 300 ft. greater length of line. Find Vertical 3 on the base line representing the economic value of 1 ft. of distance; trace the radial thence up to the concrete-

lined conduit line, and read 289 ft.; add this to the 1 500 ft., giving 1 789 ft., the virtual length of Route No. 2 expressed in terms of concrete-lined conduit. Then find 2 on the tunnel line (top of Fig. 3), trace the radial down to the lined conduit line, and read 1 324 ft., which is the virtual length of Route No. 2 expressed in terms of lined conduit. A comparison of this value with that of 1 789 ft. for Route No. 2, decides in favor of Route No. 1. Mr. Hawgood.

M. F. STEIN, ASSOC. M. AM. SOC. C. E. (by letter).—Referring to Fig. 1, the writer notices that in the "Cement-lined Canal," "Concrete Flume," and "Tunnel" sections, the author has used a thickness of concrete of 3 in. It would seem, from the writer's experience and observation, that though this thickness may be theoretically sufficient to withstand the water pressure, it may not give a sufficient factor of safety for various contingencies which cannot be calculated. It is generally difficult to obtain water-tight construction with a thin wall, using the customary type of labor and superintendence in mixing and placing the concrete, and frost action may prove destructive in a cold climate, especially in the event of seepage through the concrete. A thin section, furthermore, provides little safety against cracking, should there be any unequal settlement. These points would apply with less force in a warm and arid region, but the author does not limit his discussion to such conditions. Mr. Stein.

As regards the tunnel lining, though this may be sufficient for very firm rock, where it is only desirable to obtain a smooth surface, for hydraulic reasons, it would not offer much resistance to the rock movements, shelling, and thrusts experienced in limestone, shale, or clay formations. This is exemplified in the recent failure of the Montreal conduit, under an apparent pressure much less than might be expected in certain kinds of rock, even though the minimum thickness of concrete was 8 in.

Although these arguments affect the diagram (Fig. 2) to some extent, they have, in the writer's opinion, a more important significance. Papers and discussions in the *Proceedings* are widely copied by engineering periodicals, and any diagrams, especially if dimensioned, are taken at face value by many engineers, outside of the Society, some of whom, by training and experience, are incapable of exercising proper discrimination. To such, the sections of Fig. 1 would make an especial appeal, owing to their apparent economy and simplicity, and might be applied without regard to their suitability to conditions which are not those met in average practice.

Relative to this, the writer recalls the failure of a small dam, about 15 ft. high, the design of which was copied from a textbook illustration of a well-known high arch dam, all dimensions being reproduced *pro rata,* with very illogical and inadequate results, further accentuated by the fact that though the original was built in rock, the copy rested on clay.

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CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN.

Discussion.*

By A. W. Buel, M. Am. Soc. C. E.

A. W. Buel, M. Am. Soc. C. E. (by letter).—Mr. Gregory, Mr. Janni, and others have brought out most of the points involved, but there remains one of vital importance as affecting the comparison of the hinged and fixed-arch rib as made by the authors. Although they have used the proper form for their hinged ring, their fixed ring is neither correct nor economical; and, therefore, is not a fair comparison. Being segmental, it is subject to great moments at the springing line, as the ring does not conform to any pressure line for any practical conditions. Mr. Buel.

The only economic justifications for the three-hinged ring are difficult foundation conditions and a rise of less than one-sixth of the span with a possible yielding of the abutments.

The authors exaggerate the difficulty of computing the fixed arch, for, by systematizing and tabulating the work, it is quite possible to make the computations in 2 days or less. Moreover, it does not seem to be good engineering practice, nor creditable to the industry of the Profession, to advocate a design because it may save a few hours or even a few days of work for the designer. This is aggravated when there is room for the suspicion that any such saving in the designer's time and energy is largely due to his unfamiliarity with fixed-arch computations.

The advantages of the ribbed ring have been demonstrated and advocated† by the writer in designs worked out in 1899 and 1901.

* Continued from January, 1914, *Proceedings*.
† "Reinforced Concrete," First Edition, 1904.

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THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.

Discussion.*

By Messrs. Charles Rufus Harte, W. Kiersted, and George B. Stone.

Mr. Harte.

Charles Rufus Harte, M. Am. Soc. C. E.—Papers such as Mr. Alvord's, which by stimulating discussion tend to spread a broader knowledge of the important subject of valuation, are particularly valuable at this time when the situation seems not unlike that which led the old New Englander, not in sympathy with the party then in power at Washington, to add to his prayer for Divine guidance for the then President the earnest statement "Thou knowest, Lord, he needs it." At the same time, it would seem not entirely out of place here to raise the question whether, in the treatment of the subject, many of us are not beclouding the facts by analyses and discussions which are too academic.

The author's definition of "depreciation" includes four widely diverse items of loss of value:

1.—Organic depreciation—the loss of value because of wear of the substance of the elements which make up the machinery or plant—begins with the life of the plant and continues practically uniform to the end. Plotted in terms of time and loss, it is very close to a straight line.

2.—Functional depreciation—the loss of value because elements of the plant get out of correlation—begins with the life of the plant, and, particularly if not met by proper maintenance, accelerates very rapidly until near the break-down point, when it checks and then con-

*Continued from January, 1914, *Proceedings*.

Mr. Harte.

tinues slowly to the end. Proper maintenance brings back conditions, but not quite to what they were at the previous take up, so that, plotted in terms of time and loss, functional depreciation appears as a saw-toothed curve.

3.—Obsolescence—the loss of value because the product of the plant no longer meets market requirements as to character—may attack a property at any time, or may not occur at all. Occurring, it may progress with great rapidity; it may progress slowly; or it may progress, halt, and progress again; so that its curve, in terms of time and loss, may be of almost any form. A striking instance of acute obsolescence is that which, almost over night, changed the cable traction system of the Broadway street-car line in New York City from the latest word in traction to a practice not merely obsolescent but actually obsolete. Thus, also, the rapid development of the alternating-current lighting system with high-potential primaries feeding through transformers, the low-potential distribution circuits, in an almost equally brief time made the Edison three-wire underground system practically of the past.

4.—Inadequacy—the loss of value because the product of the plant, though meeting the requirements as to character, is of insufficient quantity—is, like obsolescence, uncertain as to its occurrence and variable as to progress, though not to the same extent as the former; and, like it, its curve, in terms of time and loss, may be of almost any form.

These four elements are so different in character that it is hardly possible to treat them properly together. If the system under consideration is of such extent that the average of conditions is a fair general average, life tables may be applied with very good results, so far as organic and functional depreciations are concerned. Local conditions, however, often make the system average quite different from that of the tables, with corresponding error unless proper correction factors are used. Obsolescence and inadequacy, at best, can only be guessed at.

The author truly says of "the reproduction method, or cost new less depreciation," "This line of evidence is only one source of information as to value." The fair value of the property used and useful for the public is neither the cost to produce nor yet the cost to reproduce; it is the capitalized earning power. It has been argued that for rate-making the earning power cannot be considered, as it is a direct function of the rates to be regulated, and it is hard to escape the conclusion that—at least where there is competition—rate-making must of necessity be an arbitrary procedure. Economic laws, more powerful even than governmental commissions, will compel the practical equalization of rates, even though different values of competing utilities would otherwise permit them to charge different rates for the same service.

Mr. Harte.

In determining physical value, the Courts almost universally—if not entirely so—have held that depreciation must be considered. It is difficult to find logical grounds for doing otherwise. Use—and for that matter disuse as well—causes losses of quality, quantity, and, if extended, of efficiency. That these losses ought to be made good by the owner, or that there exists a fund dedicated to such making good, in no way changes the facts that the loss has occurred, although such outside conditions may offset the physical loss and result in an unchanged or even enhanced value when the two are considered together.

With the proposition to determine past depreciation "on the basis of a sinking fund" there will be much disagreement. Past depreciation is a fact which, except as to its rate of progress, is usually ascertainable without great difficulty, and it should be actually ascertained if the determination is to stand investigation.

A depreciation fund properly secured may be included as one of the physical parts of a property, and, if equal to the amount of depreciation, will, of course, offset it, but in such case the depreciation itself must be determined in order to see if the fund is adequate; it is equally possible, unless adjusted frequently, that it may be greater or less. Nor is the writing off of such an item on the books without dedication of the actual money merely an opinion. Properly written off, it stands as a liability of the property, reducing by its amounts the net earnings of the prosperous plant, or adding to the deficit of the losing venture.

Under the caption "Utilities Commonly Require Growing Plants" the author states that extensions produce a developing series of increments, each with its own depreciation. Although this is literally true, the situation is far less complex than the statement would indicate, because much of an extension merges into and thereafter is treated as a part of the older installation, and depreciation is taken as an average on the whole. Building extensions of comparatively small amount, additions to piping and wiring systems, all such are adaptable to this treatment, leaving only the larger individual items to be dealt with separately.

It might well have been said that the earning power of a partly worn plant is almost invariably greater than that of a new one. Smooth operation, and the maximum efficiency, are had only after the very appreciable physical losses due to what Mr. Wilgus, in his recent paper "Physical Valuation of Railroads,"* aptly called the "educational period."

The author states that "One of the reasons the Courts have for favoring the reproduction method, or cost new less depreciation, lies in the fact that such procedure includes appreciation as well as excludes depreciation." It is probable that the attitude of the Courts

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

Mr. Harte.

in this matter is due more to the logic of the procedure than to a desire to include appreciation.

The cost of educating the public to the value of the utility, though not attaching to the property in that particular form, remains as a very considerable asset in the going value, which is due to "a fundamental conviction that a utility is really necessary and useful."

In "The Practical Treatment of Depreciation Accounts" the author apparently considers safe only that fund established with some trust company. Aside from the fact that trust companies have failed, and doubtless will do so in the future, such companies find the necessary monies to meet fund interest, overhead charges, and profit, in other securities, and, if the utility by direct investment can save the middleman's profit and overhead charges, and thus reduce the investment necessary to produce the required result, it would seem but fair thus to permit; the matter is purely the question of fact as to whether or not the "securities" are such in more than name, and whether they are a part of the property, or have passed out of it. The first calls for expert financial knowledge; the second must be evident from the books.

A reasonably safeguarded fund or investment for renewal purposes, in a total valuation, should be considered as much a part of the property as the material which it will presently replace. Such replacements for the most part can be made economically only after considerable wear and consequent loss in physical value. If this physical loss is not excessive, and is properly set up on the books as a liability, it is obviously unjust to deprive the owner of its proportion of earnings merely because of its changed form.

The author's first conclusion, that a trust fund to meet depreciation if "properly computed and accurately kept"—this whether in this connection "kept" has reference to the bookkeeping or the actual holding—should be considered a part of the property "used and useful for the public," would seem obvious.

The second conclusion, that the fund must be "actually in hand," is also obvious, if a reasonable interpretation is put upon "in hand." To say, however, that proper book entries do not constitute such condition is, to put it mildly, unfair; carried to its logical conclusion, such reasoning would preclude the consideration, as assets, of any unpaid accounts.

The third conclusion, that "past depreciation may be computed accurately on the sinking-fund basis," is only warranted when "all the facts of the past are known," in which instance there would seem to be little occasion to compute what is already had. It should not be forgotten that, in the Minnesota Rate Case, there were thrown out of Court, because obtained by hypothetical assumptions, figures for land values which were undoubtedly very close to the facts. For the future,

knowledge of the past will often establish correction factors to apply to and make applicable life tables, so that organic and functional depreciation can be predicted with reasonable closeness, but the possible effects of obsolescence and inadequacy can only be guessed at, in the light, respectively, of the development of the art and the local conditions. Mr. Harte.

Conclusion four, that depreciation accounts should be kept by groups having similar life characteristics, and should be revised from time to time, is good, and might well call for considerable detail in the groups. It should be always remembered, however, that such accounts are only estimates, the relations of which to the facts can only be known by actual comparison.

Conclusion five, as stated, that the use of a reserve or sinking fund for private gain precludes its consideration as a factor of value to the utility, is sound only as it applies to such portion of the return from the fund as becomes "private gain." The use of funds set aside for the specific purpose of a reserve or sinking fund in the purchase of securities or in other forms of loan does not destroy the asset unless the security therefor, whether bonds, stocks, or personal notes, loses value; nor if such security pays a rate of interest higher than necessary for the calculated fund, and this excess be withdrawn and otherwise used, does it impair the protection of the remaining portion.

W. KIERSTED, M. AM. SOC. C. E. (by letter).—Mr. Alvord's interesting paper on depreciation appears to be largely a discussion of the paper by Mr. Grunsky,* or at least to have been suggested by the views set forth in that paper. Mr. Kiersted.

In this discussion Mr. Alvord raises four distinct questions, namely:

Shall allowance be made for depreciation of physical structures in computing value of a public utility property by the reproduction method for rate-making?

Should a sinking fund, created to take care of depreciation, be allowed to earn a return—in fact, to earn the same rate of return as other portions of the property?

Should a sinking fund thus created be considered as a part of the property?

Should the units of property be grouped according to their respective life limit in accounting?

A difficulty at once arises in discussing these questions from the fact that the viewpoint of the author is not made perfectly clear. One can readily conceive of lines of procedure under the reproduction method of computing value which might admit of opposite answers to all the questions mentioned. Under the circumstances, the dis-

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

Mr. Kiersted.

cussion of these questions is likely to develop divergent opinions, depending on the viewpoint of the writer.

The reproduction method of valuation seems to require more hammering to reduce it to proper form and more clearly to restrict or fix the line of procedure; and, until this is done, there is little hope of arriving at a conclusion on questions of the kind raised by the author, especially when considered somewhat abstractly. In many instances computations relating to the rate of return on values found by the reproduction method have not sufficiently discriminated between a fair rate of return on the value of the depreciable and the non-depreciable property, on the value of the tangible and the intangible property, and on the value which is actually earned and that which includes an unearned increment. Such questions as these seem to have been overlooked in most of the rate cases of recent years. Moreover, most of the units going to make up the value of a public utility property possess no value except for the particular purpose for which the public utility is used, though other units possess an intrinsic value of their own, entirely apart from and independent of the public utility with which they are connected. Investment in units of one class is obviously more hazardous than in the other class, and there should be a corresponding difference in the rate of return to which the respective classes of property may be entitled. The questions raised by the author fall in a similar category, and unless they can be discussed thoroughly and comprehensively in connection with all other questions which enter into the computations of value by the reproduction method, or by any other method, no conclusive answer can be given to them.

The writer's discussion will be brief, and directed largely to simplifying, rather than complicating, the questions propounded for discussion.

Several valuations of water-works properties by commissions of experienced men, for the purpose, among other things, of rate-making, have been carefully analyzed and platted for the purpose of seeing how the methods of valuation therein used would work out in practice. However, only that part of one of the analyses which seems to apply to this paper, will be considered. The value of this particular property was computed by the commission in accordance with the reproduction method, as used in many of the late valuation cases, and allowance was made for depreciation as well as for enhancement of value and for unearned increments of value. Depreciation was computed on the sinking-fund basis by assuming a life for thirty or more units going to make up the physical property, with sinking-fund increments uniformly bearing interest at the rate of 3½% per annum, regardless of whether the unit under consideration was short-lived or long-lived.

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Under the assumptions made in the writer's analysis, the composite property might or might not be assumed to have a definite life, but, for the purpose of a part at least of this discussion it is considered as possessing definite life—a life limit having been assumed for all the units of the depreciable property.

On the diagram, Fig. 1, are platted two curves, each point in each curve being platted with reference to the average age and equivalent annual percentage rate of depreciation of the composite property. One, the full-line (upper) curve, is for convenience termed a definite-life curve, the other, the broken-line (lower) curve, is termed a perpetual-life curve. Average age is computed in the usual manner by the weighted method. The equivalent annual percentage rate of depreciation is found by dividing the total depreciation, as of the date under consideration, by the average age of the composite property, as of the same date. Whenever a break occurs in either of these curves it indicates that some one of the many units considered has reached the limit of its assumed life. It is assumed that throughout the whole period under consideration the total investment in the physical property remains the same. This assumption is met in the analysis by making two computations whenever any unit, going to make up the composite property, reaches its life limit. One computation consists of determining the average age and the total percentage depreciation of the composite property at the time when a unit reaches its life limit and is depreciated 100%; the other computation consists of determining the average age and total percentage depreciation as of the same year and date as the preceding computation, taking into consideration the new or substitute unit as of the same class and cost as the replaced unit. The new curve, starting with the substitute unit, continues unbroken until a second unit reaches its life limit, when another break occurs and two similar computations are made and a new curve started, and so on until all the units have been thus considered, making what is for convenience termed a cycle.

During a cycle of this particular property the number of units replaced by new ones of the same kind and cost are as follows: four units four times, eight units three times, nine units twice, and ten units once.

These curves serve no particular purpose except to illustrate the variation of the equivalent annual percentage rate of depreciation of the composite property during different periods of a cycle. They indicate a progressive and regular increase of the equivalent average annual percentage rate of depreciation until units begin to disappear, which is characteristic of the sinking-fund method of computing depreciation. The computations made in connection with the analysis lead to the following suggestions:

Mr. Kiersted.

First, the definite-life (full-line) curve:

a.—The total percentage cost of replacements during a cycle is 132.5% of the total investment in the physical structures composing the composite property, equivalent to the amount of an annuity of 0.67% of the investment in physical structures, earning interest in a replacement fund at the rate of 3½% per year during an average age period of 63 years.

b.—The value of the investment at the close of the cycle is the same as at the beginning of the cycle, the replacements having been made promptly as each unit reached the limit of its assumed life period.

c.—In order that the annuity of 0.67% of the investment in physical structures may amount to the cost of replacement during a cycle, the annuity and interest increments accruing thereon must remain undisturbed in a replacement fund during the entire cycle.

d.—If the owner of the property allows the replacement fund to remain intact, and uses new money to make replacements, then at the expiration of a cycle his property is worth 100% plus the monies in the replacement fund.

e.—If the replacement fund accumulates in the manner before stated and remains intact, and the owner makes no replacements with new money (assuming such a condition to be possible) then at the expiration of a cycle he has no property, but is the owner of a large portion at least of the monies in the replacement fund.

f.—If the owner of the property makes replacements with money taken from the replacement fund, as units need to be renewed, then such monies as may be taken from the replacement fund cease to draw interest as a part of that fund, but earn, as a part of the plant, their *pro rata* of the fair return due the owner and of the annuity going into the fund. The result is a deficit in the replacement fund when computed as per "*a*" and "*c*", of a large amount at the end of a cycle. In order to prevent such a deficit, and at the same time to permit monies to be taken from the replacement fund for renewals of plant as conditions demand, the annuity should be about 0.95% of the investment in physical structures.

g.—Throughout the entire cycle, computations of the fair return and of the annuity to the replacement fund should be based on 100% value of the investment.

h.—Replacements having been made to the property as needed, and the property thereby maintained at 100% value, there is no need of a sinking fund to return the original investment, because the property possesses the characteristics of perpetual life.

i.—Assume the owner to sell the property at some time during a cycle, the replacements having been made from monies accumulated in the replacement fund, then the property would be worth to the

Mr. Kiersted.

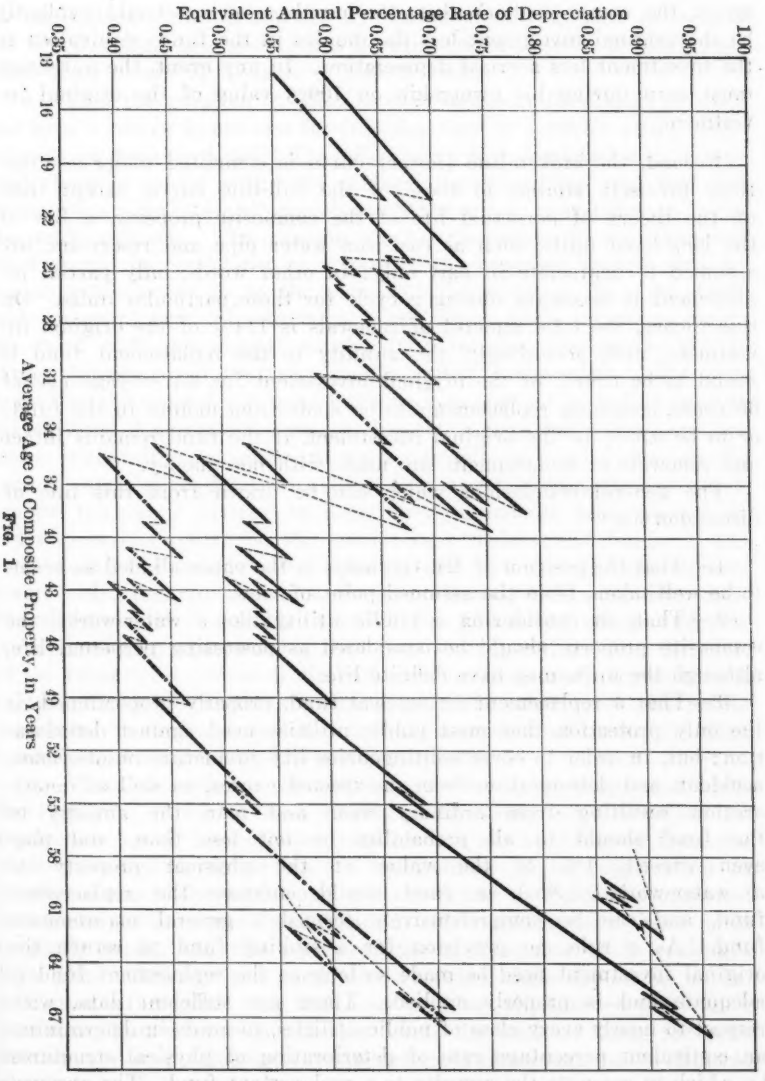

FIG. 1.

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purchaser the full amount of the investment, provided the remaining monies in the fund go with the property. If the monies in the fund go to the owner (seller) then the purchase price should evidently be the original investment less the monies in the fund—equivalent to the investment less accrued depreciation. In any event, the purchaser must earn during his ownership on 100% value of the original investment.

Second, the broken-line (lower) curve is computed under assumptions precisely similar to those of the full-line curve, except that, on the theory of perpetual life of the composite property, a few of the long-lived units, such as cast-iron water pipe and reservoirs, are assumed to depreciate in part only; in other words, only partial replacement is necessary during a cycle for these particular units. On this theory, the total amount of renewals is 114% of the original investment, and, accordingly, the annuity to the replacement fund is found to be 0.80% of the original investment for an average age of 65 years, assuming replacements to be made from monies in the fund; or to be 0.53% of the original investment, if the fund remains intact and renewals or replacements are made with new money.

The general conclusions which can be drawn from this line of discussion are:

1.—That the position of Mr. Grunsky, in the paper alluded to, seems to be well taken, from the assumed point of view.

2.—That, in considering a public utility like a water-works, the composite property should be considered as possessing perpetual life, although the units may have definite life.

3.—That a replacement or renewal fund, properly proportioned, is the only protection that most public utilities need against deterioration; but, in order to cover contingencies like premature obsolescence, accident, and deterioration from unexpected causes, as well as deterioration resulting from ordinary wear and tear, the annuity to the fund should in all probability be not less than, and may even exceed, 1% of the value of the physical property of a water-works. Such a fund would embrace the replacement fund, and can be comprehensively termed a general maintenance fund. As a rule, no provision for a sinking fund to return the original investment need be made so long as the replacement fund is adequate and is properly applied. There are sufficient data, with respect to nearly every class of public utilities, to guide in determining an equivalent percentage rate of deterioration of physical structures by which to compute the annuity to a replacement fund. The annuity itself will doubtless vary for different properties of any given class. The annuity to a fund should be considered as earning interest at

sinking-fund rates, the interest being considered, of course, as a part of the fund. None of the increments to a replacement fund should be capitalized. The monies in the fund should be used as needed for replacements, and should not be surrounded by a wall which prevents their proper application in that direction. Mr. Kiersted.

4.—That the question of whether a replacement fund shall or shall not earn a return is not one for consideration by a public utility commission in definite detail. The office of such a commission is to see that there is a fair return for rendering good service, and that the properties are maintained in such condition and so operated as to serve the public properly. The question whether the monies in a replacement fund should be considered as a part of the property for use only in replacements or to go to the owner of the property, is a debatable and important one. Should it go to the owner, he is obliged to credit the fund with the regular sinking-fund interest increments, but it cannot be conceived that any public utility commission would seek to destroy the incentive to good business management by ruling against the earning of a return on monies in such a fund, so long as replacements with full-value units are made promptly as needed; nor should such a commission delay in calling to account a management which ignores the public interest by delaying replacements when needed, in order to earn a return on the monies in a replacement fund. If the fund remains attached to the property and is to be used for replacements only, the annuity must be proportioned with some liberality, in order to avoid, if possible, a deficit; but if a deficit occurs and new money goes into replacements, then the owner is entitled to a re-rating of the property, a return of the value of his new-money replacements, or an increase of capital investment. On the other hand, the public is entitled to similar consideration under reverse conditions. In other words, properties must be re-rated periodically.

5.—That depreciation should be considered in connection with the valuation of the physical structures of public utilities on the reproduction basis. Depreciation may or may not be deducted from the value thus found, depending on the manner of the application of monies in the replacement fund and the extent to which the unearned increment enters into consideration. If replacements have been promptly made, and the policy of the management is to maintain high plant efficiency and good public service, there should be no depreciation of original cost value of existing physical property in a rate case; but, where reproduction forms the basis of value for rate-making, and such value includes an unearned increment, due to changed physical conditions or enhanced values of certain units resulting from city environment, and the same rate of return is allowed on these elements of value as on other portions of the property, then depreciа-

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tion should be allowed in computing fair return; but if the enhanced values and unearned increments of value were put in a class by themselves and were allowed such a rate of return as may be equitable, then depreciation of the physical property might be as consistently ignored in a rate case based on reproduction as when based on original cost value.

6.—That, in accounting, there is no necessity for dividing a property into groups of units according to some assumed life and charging each account with depreciation in accordance with some mathematical rule. The property should be considered as a whole, or in groups arranged by structural classification, deducting from the values thus recorded in the accounts the cost or value of any article or unit that is abandoned or replaced, and adding to the account the substitute article or unit. In doing so, proper discrimination should be drawn between minor renewals belonging properly to the operating account and replacements which actually affect the investment. Every engineer must concede that the place for working out details and for clarifying and simplifying any plan or specification is in the office, in order to render the work of the field parties as simple, clear-cut, and free from the danger of error as possible, and it can be conceived that similar principles should apply to accounting.

Although much more can be said on the questions under discussion, if considered in their relation with the general subject of valuation, and from a practical rather than an academic point of view, the writer is not inclined to extend the range of discussion beyond that laid down in the paper, except to add one or two remarks, particularly as the writer's discussion of Mr. Grunsky's paper presents relevant views which Mr. Alvord seems to indorse, in part, at least.

The writer has stated previously that the reproduction method of valuation needs more hammering to reduce it to proper form for general use. The attempt to apply rules to valuation work, with mathematical precision, is certainly a mistake, and leads the adherents of these rules into error in matters which necessarily involve the use of an enlightened and experienced judgment unfettered by precedent or rule. So many times the "yard stick" of value (to use the term of another) in contact with a mind overheated with the discovery of some element of value hitherto disregarded, or with some new idea, has been so expanded as to distort a line of reasoning believed at the time to proceed from an elevated or progressive point of view; or, on the other hand, the "yard stick," with the middle half cut out and the ends of the remaining portion joined with unaltered dimension at either end, has been used as though of up-to-standard length. There is no possible way of reconciling the differences thus encountered, or of bridging the wide gap exposed in the deductions thus made, except

by going back and standardizing the "yard stick." In other words, fundamental principles must be agreed on, and these principles must be applied consistently, intelligently, and cautiously, and with due consideration of the character, condition, and individuality of each property to be valued. In no two instances will the application of the principles be the same, but the results of the work of two or more appraisers should not vary so much as to be irreconcilable, or as to cause confusion of mind for those who hope to be profited by the expert's wisdom. Mr. Kiersted.

GEORGE B. STONE, ASSOC. M. AM. SOC. C. E. (by letter).—The question of depreciation of public utility corporations is of great moment at the present time to all members of the Profession, and especially to engineers engaged in the work of valuation of public service corporations, and is a subject of vital importance to both the public and the financial interests. Mr. Alvord's paper is of great interest, and he has presented his view of the question clearly and concisely. To the broad discussion that must take place before we finally come to an agreement, his paper is a valuable contribution. Mr. Stone.

The writer's experience has been entirely in railroad work, and in the remarks that follow he has confined himself to that point of view.

The valuation of common carriers, as outlined by the Federal law, gives a suggestion of "the cost of reproduction less depreciation" and the question immediately arises: "What shall this depreciation be, and how shall it be disposed of? The writer cannot agree with Mr. Alvord that depreciation is the same for all purposes of valuation, and will attempt to outline the reasons.

Depreciation for Rate-Making and Fair Return.—W. J. Wilgus, M. Am. Soc. C. E., in his paper, "Physical Valuation of Railroads,"* stated the question of depreciation for the purpose of rate-making very clearly and convincingly from the standpoint of the accountant working with an engineer on this subject, and the writer agrees absolutely with those views. The writer believes, however, that there are still other and equally important reasons why depreciation, as the word is commonly taken, should not be deducted from the cost of reproduction new, as of the date of the valuation. Stripped of all theories and technicalities, valuation for the purpose of rate-making becomes a question of allowable rates for a given service due to the corporation from its patrons. The Federal law suggests that these rates shall be determined on a basis of fair return based on the cost of reproduction new less depreciation, but it does not state how this subject of depreciation shall be considered, and has left unsettled a question which is open to much discussion and controversy. The object of the law was to furnish a means of settling many of the disputes that have arisen

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 208.

Mr. Stone.

over the question of rates in the past, and to prevent and settle all such controversies in the future. Such being the case, the consideration of depreciation for rate purposes becomes one of service, and not physical value, unless the service of a given property has fallen below that which might reasonably be expected from a given investment. If it is found, in the valuation of a given property, that the service is less than might reasonably be expected from a given investment, then the property should be assessed a depreciation charge equal to the amount of depreciation from this source. This applies to a well-managed, going property, for if a property has been poorly managed, and there is evidence of needless expense and unwarranted waste, no valuation based on the cost of reproduction of its physical parts would ever give a correct value for rate-making or for any other purpose except that of taxation. Many properties will come between the extremes, but the principle remains the same.

Depreciation for Taxation.—The law, as practised, in regard to the taxation of all property, recognizes but one value: the amount that a given property will bring at forced sale; and, for taxation purposes, depreciation assumes an entirely different meaning from that for rate-making; in that it becomes a recognizable physical value, it becomes a problem of actual worth as shown from the inventoried cost new less any depreciation value due from any cause that affects its physical condition. It is a problem of actual physical worth, and cannot include anything but its true dollars-and-cents value. "A" owns a certain lot of land for which he paid $500 and can sell it for this amount at any time that he chooses, but, by waiting a little, he can sell it for $1 000. The $500 is the quick-sale or taxable value, and the larger figure is the sale value. This principle of taxation has been generally recognized by the Courts in taxation cases. The general principle of this subject is very simple, and the writer fails to find any reason for complicating it by the adoption of assumptions that do not exist in every-day practical railroad management; nor does he believe that any valuation based on theoretical assumptions will stand in the Courts. Depreciation for taxation can be the loss due to physical age of a property and nothing else, under the present construction of the tax laws.

Depreciation as Applied to Sales Value.—This phase of the subject is very closely allied to that of taxation, but has many features that prohibit the combination of the two under one head; it is also closely allied to depreciation for the purpose of rate-making, in that it has a value which is dependent on service. There are so many phases to this portion of the subject that the writer does not believe that, at this time, it would be either good business or good engineering to attempt to lay down a general law on the subject; nor, at the present stage of the matter, could a satisfactory general law be laid down. Every

Mr. Stone.

carrier has a different problem to solve when it comes to the placing of a sale value on its property. The appraiser has the simplest portion of his work complete when he has finished the physical inventory of the property and made the proper deductions due to the physical depreciation of the property as of any certain date. He has many questions to ask himself: Is the location as it stands the best to be found, considering the traffic? In case the line were sold, would the purchaser rebuild any portion of it for economic reasons? Are there any competing lines built on a more economic basis which affect the earning power of the road under consideration? All these, and many others of a like nature, must be answered equitably before depreciation for sale value can be determined. Such questions as these have values of depreciation, and the appraiser must determine the true values and consider them a part of his appraisal. One road may not have cost double what its rival did, and yet the service of both roads is the same. There is a definite depreciation value here, chargeable against the more costly road, in case an appraiser was placing a sale value on it; but, another critic will say, there would be an appreciation value in favor of the cheaper road in case a sales value was to be placed on it. This is very true; there is a question of which value to apply, but that is for the appraiser to settle. The main point that the writer wishes to make is that there are values of depreciation for sales purposes which do not apply to either the taxation or the rate question.

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PAPERS AND DISCUSSIONS

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REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.

Discussion.*

BY MESSRS. J. K. FINCH, ALEXANDER POTTER, AND CHARLES B. BUERGER.

Mr. Finch.

J. K. FINCH, JUN. AM. SOC. C. E. (by letter).—In some cases reinforced concrete has been substituted for steel in the construction of tanks and stand-pipes, because it was believed it would result in economy, both in first cost and maintenance, and because such structures are generally on high ground, where they can be seen from afar, and it was hoped that a more satisfactory architectural effect could be secured by its use. The first of these expectations has generally been realized, but not the latter, not on account of the material, which is admirably suitable for architectural treatment, but because the failure to obtain a truly water-tight construction causes the marring of the surface of the structure by the formation of moist spots, due to "sweating," and patches of efflorescence. It has been pretty well established that the chief difficulties are:

1. Poor concrete;
2. Vertical cracks in concrete due to excessive stretching of steel under hoop tension;
3. Horizontal cracks between days' work, due to difficulty of joining old and new concrete, and the fact that concrete shrinks in setting and the successive layers are held apart by the vertical steel necessary for supporting the hooping rods; and
4. The formation of a horizontal crack, about 5 ft. above the base, in all cases where the bars in the floor or base have been turned up into the side-walls.

*This discussion (of the paper by Edward Flad, M. Am. Soc. C. E., published in December, 1913, *Proceedings*, and presented at the meeting of February 4th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Finch.

The remedy for the first is good materials and workmanship. A solution of the second difficulty has been proposed by H. B. Andrews, M. Am. Soc. C. E., in a paper entitled "A New Theory for the Design of Reinforced Concrete Reservoirs,"* presented before the Boston Society of Civil Engineers in 1910. This is the method followed by Mr. Flad in the design of the St. Louis tank. The writer desires to call attention to the fact that Mr. Flad, however, has used a working tensile strength for the 1:1½:3 limestone concrete of 290 lb. per sq. in., which appears to be excessive. It must be borne in mind, of course, that economy demands the use of a low factor of safety, say 1½, for the concrete, which is ample, as the ultimate safety of the structure depends on the steel, but this figure, 290 lb., has been taken apparently from Mr. Andrews' paper, in which he proposed this stress for a 1:1:2 concrete, not a 1:1½:3 mixture.

It is unfortunate that there are so few tests of concrete in tension on which to base a satisfactory working stress. The tests that have been made show a considerable range in values, due to the effect of different aggregates, methods of mixing, etc., and, generally, only unreinforced specimens have been tested. M. Considère made a number of experiments on reinforced concrete in tension, and concluded that the action of the reinforcing steel was to distribute the stress throughout the concrete and enable the latter to take a tensile loading considerably higher than the plain unreinforced material. It was shown later, however, that this was not proved by the tests, as M. Considère's method of computing the stress in the concrete and steel was incorrect. In fact, it was shown that a reinforced specimen developed cracks, and the concrete practically failed at about the same load as an unreinforced specimen. A few tests have been made at Columbia University on 6 by 6-in., 1:3:5 briquettes reinforced with 0.68 to 4.26% of steel, which showed a failure stress in the concrete, based on the break in the stress-strain curve of 205 lb. per sq. in., which is about the same as would be expected from an unreinforced specimen.

Mr. Andrews obtained his figure, 290 lb. for 1:1:2 concrete from a few tests he had made, which gave an average of 281 lb. per sq. in., by increasing his test value by 25 and 10%, which gave him 386 lb., which he considered as the ultimate strength for a large reinforced specimen, and that 290 lb. was sufficiently lower than this to be used in the design. The increase of 25% was to allow for "the usual increase in strength of large size over small specimens," his test pieces being 4 by 4 in. in cross-section; the further increase of 10% was to allow for the effect of reinforcing steel, and was probably based on M. Considère's deductions. Neither of these increases seems to be justified by any tests yet made, but Mr. Andrews' tests did give rather

*Journal, Assoc. Eng. Soc., Vol. XLVI, 1911, p. 391.

low values. The following stresses are estimated to be fair values for the quality of concrete used in tank work: Mr. Finch.

Mix.	Age, in days.	Ultimate strength, in pounds per square inch.
1:1 :2	30	300-400
	90	400-500
1:1½:3	30	250-350
	90	350-450
1:2 :4	30	200-300
	90	300-400

The working stress should depend on the length of time to elapse before the tank will be filled, but 290 lb. per sq. in. appears to be high for a 1:1½:3 mixture.

There seems to be no remedy for the difficulty due to field joints, except pouring in one operation, which should be done whenever possible. Proper care in cleaning the surface of the old concrete, and the use of copper strips or dams, will doubtless do much to remedy this trouble.

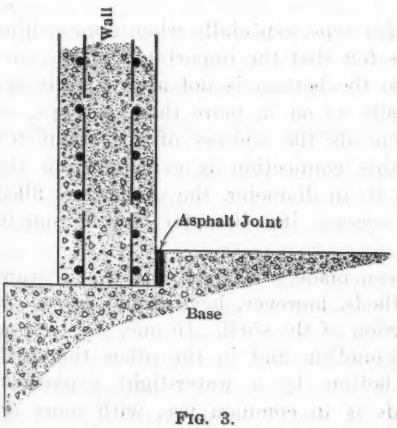

FIG. 3.

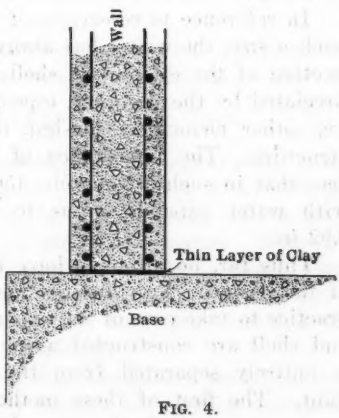

FIG. 4.

The formation of a horizontal crack, in cases where the bottom reinforcement has been run up into the sides, is due to the fact that the lower part of the side-walls of the tank is prevented from stretching under the hoop tension, due to the restraint of the bottom reinforcement. Indeed, the writer is of the opinion that in the lower part of Mr. Flad's design the wall acts as a cantilever in carrying the water pressure, and little or no hoop tension exists. He cannot see why this method of carrying up the bottom reinforcement has been followed in most cases. There is apparently no reason for not designing a tank simply as a vertical section of pipe resting on, but independent of, the base, as shown in Figs. 3 and 4. The pressure on the joint between

Mr. Finch. the sides and base should be investigated, of course, for the effect of wind and also for possible upward water pressure, but, in the majority of cases, this joint will always show an ample positive pressure.

The writer has used the form shown in Fig. 3 in preparing a design for his students in reinforced concrete, and has since learned that Mr. P. D. Johnson, Hydraulic Engineer of the Ontario Power Company, Niagara Falls, has made tests of the type of joint shown in Fig. 4, and has actually constructed the No. 2 Surge Tank of the Ontario Power Company on this principle. This tank is 70½ ft. high and 79 ft. in diameter, and no cracks have developed. The writer understands that a thin coating of clay was placed on the base under the walls before the wall concrete was deposited.

Mr. Potter. ALEXANDER POTTER, ASSOC. M. AM. SOC. C. E. (by letter).—As far as the writer knows this is the largest reinforced concrete reservoir in America of a type in which the hydrostatic pressure is resisted by ring tension; therefore it is of more than usual interest to the Profession.

In reference to reservoirs of this type, especially when approaching such a size, the writer has always felt that the importance of the connection of the cylindrical shell to the bottom is not always fully appreciated by the designer, especially as on it, more than, perhaps, on any other element of design, depends the success of the completed structure. The importance of this connection is evident from the fact that in such a reservoir, 150 ft. in diameter, the shell when filled with water expands so as to increase its diameter approximately 0.92 in.

Thus far, no attempts have been made to standardize this feature of the design. Two distinct methods, however, have been evolved in practice to take care of the expansion of the shell. In one, the bottom and shell are constructed as a monolith, and in the other the shell is entirely separated from the bottom by a water-tight expansion joint. The first of these methods is in common use, with more or less success, and is the one which seems to have been used successfully in constructing this settling tank.

Such a rigid connection, to be successful, calls for heavy steel reinforcement throughout the entire bottom, which, for economy, must be of as thin a section as possible so that, by stretching, it can readily adapt itself to the increase in diameter when the reservoir is full. This reinforced concrete bottom must be designed so that it can stretch in all directions on an average of 1 in 2 000 without developing cracks of sufficient extent to impair its water-tightness. There is every reason to believe that the elongation in the bottom is not distributed uniformly and that the bottom is stressed most in its outer portion and very little, if any, in its central portion. This unequal

Mr. Potter.

distribution would tend to give radial elongations in the outer circumference of the bottom considerably greater than the average value. It appears, however, that as long as the bottom is quite thin and is reinforced in all directions with at least 0.5% of steel possessing a high elastic limit, such a bottom can be maintained practically water-tight. Ordinarily, such a heavily reinforced thin bottom must be supported on a rigid and smooth foundation in such a way as to permit of as free expansion as possible. The bottom of the cylindrical shell must also be heavily reinforced to resist the stresses due to the bending moments set up in it by the restraining action of the bottom.

A rigid connection, such as used in the St. Louis reservoir, to be successful, is wasteful of structural material. It would appear to be more economical to use some type of water-tight expansion joint between the shell and the bottom, so as to permit the shell to expand irrespective of the bottom. This latter form of construction has already been used to some extent in circular tanks of relatively small diameter. A discussion on this point is invited, as the use of such expansion joints in tanks of large size, if proven successful, will tend to a more economic form of construction.

Another point on which there appears to be a wide difference of opinion is the proper thickness of the concrete section to be used in designing the shell. The author derives the formula, $T = \frac{pr - 9A}{12c}$ for the proper thickness of the concrete wall at any point, based on the assumption that vertical cracks are not likely to develop under a tension of 290 lb. per sq. in. in the concrete, and a ratio of 10 for the modulus of elasticity of steel to concrete. With the unit stresses assumed, the formula reduces to the simpler form that at any point the thickness of the concrete must be 42.7 times the area of the steel per inch of height of wall, thus limiting the steel reinforcement in the shell to 2.35 per cent.

The assumption on which the foregoing results are based appears to be an arbitrary one. There are any number of existing structures in which the ratio of steel to concrete exceeds 2.35%, and in which the concrete remains practically water-tight. Perhaps the most striking example can be found in cement-lined, wrought-iron pipe. In such pipe the steel reinforcement is frequently as high as 10%, and yet, under the high unit stresses to which the wrought-iron shell is often subjected in service, the cement lining possesses sufficient ductility to remain intact after years of service. Many sections of such pipe have been removed by the writer from distribution mains, the lining being found as perfect as on the day when laid. There are other structures, such as reinforced concrete pipe and stand-pipes, in which the ratio of steel to concrete in the shell exceeds 2.35%, which appear to maintain their water-tightness.

Mr. Potter.

In a circular reservoir, the concrete in the shell performs two distinct functions: first, to render the structure water-tight; and second, to resist, in combination with the steel, all stresses due to bending moments which are likely to develop in the more or less rigid structure. The thickness of the concrete lining required for water-tightness, theoretically, is entirely dependent on the density of the concrete and the hydraulic head to which it will be subjected. Theoretically, therefore, it would be possible to use a very thin concrete section, provided the concrete was dense and water-proof, and the steel reinforcement very closely spaced. It is not found practicable, however, to do this. To build a truly circular reservoir is impossible. Owing to the lack of sufficient flexibility on the part of reinforced concrete, heavy bending moments are set up in the shell, due to irregularities of shape, and these must be resisted by the concrete in combination with the steel. It can readily be shown that, with only one row of circumferential reinforcing bars, the bending moments with their resultant stresses are very great and require a heavier concrete section than when two rows of circumferential reinforcing bars are used. The writer agrees with Mr. Flad that it is advisable to limit the percentage of steel reinforcement in the shell. The rule given by him, therefore, based on an apparently arbitrary assumption, appears to be conservative, especially when two rows of steel reinforcement are used. With only one row, it may be advisable to limit the ratio of steel to concrete to 1%, especially in reservoirs of large diameter.

As most engineers are, perhaps, more interested in the general subject of water purification than in the details of reinforced concrete design, the writer takes the liberty, without fear of unduly prolonging the discussion, to remark on the tank from the operating standpoint.

To remove the settled solid matter from this settling tank, it is necessary to draw off the water and place the tank temporarily out of commission. This is likely to happen at a time when the character of the water is such that it will need treatment most, and when the entire settling capacity of the water purification plant is most needed for efficiency. Unless such tanks are cleaned out regularly, the sludge deposits will greatly reduce their settling capacity, thereby lowering the efficiency of the settling process.

The writer has always been of the opinion that a tank in which the settled suspended and precipitated matter can be removed at intervals, without interfering with the operation or efficiency of the tank, possesses considerable advantages over one which must be emptied and put out of service to remove the deposited sludge. Such a basin operating continuously has a number of advantages over one which is operated intermittently. There is considerable saving in the

size of the settling basin when the settling process is carried on continuously. This saving may amount to as much as 50% over the intermittent installation in which two basins are used, decreasing somewhat with the number of basins. When the settling basin is operated continuously, its capacity is practically always available. This is not the case with the intermittent type. Mr. Potter.

There are a number of water purification plants in which the settling tanks are operated continuously. Without interfering with the operation and efficiency of the tank, the sludge is removed daily, and while it is still in a semi-liquid condition, before it has had time to compact. Settling tanks of this type have been used by the writer at McKeesport, Pa., and Georgetown, Ky. (both water-softening tanks), and also in the settling tank at Muskogee, Okla. In all these tanks, the sludge removal is accomplished by having the bottom of the settling compartments perforated with $\frac{1}{2}$-in. to $\frac{3}{4}$-in. circular holes not more than 3 ft. from center to center in all directions. These holes discharge into an under-drain system leading to a common sump. Quick-opening valves are used to regulate the sludge discharge, and when such a system is well designed the possibility of the under-drains becoming clogged is very remote. All these plants are now in successful operation. The oldest is at McKeesport, and has been in continuous operation since 1907.

The cost of properly under-draining a settling tank so that is can be operated continuously is not excessive. For instance, the Muskogee settling basin,* constructed of reinforced concrete, 212 ft. square, 18 ft. deep, and holding 6 000 000 gal., was built for a contract price of $45 000, including all piping, valves, and under-drains.

CHARLES B. BUERGER, ASSOC. M. AM. SOC. C. E. (by letter).—The choice of a circular tension-ring type of tank, in this particular design, leads to the thought that there must be some economical dividing line such that tanks of smaller diameter are more economically built with a tension-ring wall, and those of larger diameter are most economical when built with cantilever walls. Mr. Buerger.

In actual practice the most economical style will depend largely on the particular details of design adopted, the unit stresses, the assumptions as to actions of the various parts, arrangement of metal, thickness of walls, the nature of foundation, and other varying data. There is, however, a general relation which can be established which shows what is for ordinary conditions the approximate maximum diameter for which the tension-ring type of tank is adapted.

Assume an ideal theoretical tank with tapered concrete walls of zero thickness at the top water level and a maximum thickness at the bottom, and assume that the active metal is in all cases 3% of the

* The details of construction and methods of operation are described in the *Proceedings* of the American Water Works Association, 1912.

Mr. Buerger.

cross-section of the concrete, measured from the face of the concrete to the steel. Assume that the active steel is in all cases exactly equal to the theoretical quantity needed, with no allowance for laps or bonds; assume, also, that the steel at right angles to the active metal used to counteract shrinkage or temperature stresses, or to transmit stresses, is the same for the two types. For these conditions the only unequal element will then be the area or volume of the active steel.

Taking all dimensions in feet:

For the tension-ring type wall:

$$T = p\,r = 62.4\,hr, \text{ at any point} = f\,a \quad \ldots\ldots (1)$$

where a = the area of the metal,

and f = the unit stress in the metal.

Calling H the total height of the tank,

$$A\,f = \frac{62.4\,H^2\,r}{2} \quad \ldots\ldots (2)$$

and the volume of steel per foot of tank periphery is

$$V = \frac{31.2\,H^2\,r}{f} \quad \ldots\ldots (3)$$

For the cantilever-wall type:

As before, the total pressure acting on the wall above any plane

$$= 31.2\,h^2 \quad \ldots\ldots (4)$$

The moment of the pressure at that plane,

$$m = 31.2\,h^2 \times \frac{h}{3} = 10.7\,h^3 \quad \ldots\ldots (5)$$

For the assumption of 3% of metal and a ratio of modulus of elasticity of the steel to that of the concrete of 15, calling d the thickness of the wall to the steel

$$0.80\,f\,a\,d = m = 10.7\,h^3 \quad \ldots\ldots (6)$$

$$0.03\,d = a. \quad \ldots\ldots (7)$$

$$a = \sqrt{\frac{0.40}{f}} \times h^{\frac{3}{2}} \quad \ldots\ldots (8)$$

$$d\,V = a\,dh = \frac{0.40}{f} \times h^{\frac{3}{2}}\,dh \quad \ldots\ldots (9)$$

$$V = \frac{2}{5}\sqrt{\frac{0.40}{f}} \times H^{\frac{5}{2}} \quad \ldots\ldots (10)$$

For the dividing line, at which the required steel is the same for both types of wall:

$$\frac{2}{5}\sqrt{\frac{0.40}{f}} \times H^{\frac{5}{2}} = \frac{31.2\,H^2\,r}{f} \quad \ldots\ldots (11)$$

$$r^2 = 0.000066\,f\,H \quad \ldots\ldots (12)$$

Taking $f = 144 \times 15\,000 = 2\,160\,000$

$$r^2 = 144\,H \quad \ldots\ldots (13)$$

$$r = 12\sqrt{H} \quad \ldots\ldots (14)$$

Mr. Buerger.

The general relation appears then that the maximum economical diameter of the tension-ring type of tank is 24 times the square root of the water depth. This relation will be modified in actual designs by several elements not considered in the foregoing. For instance, the quantity of steel to bond the cantilever wall into the floor of the tank has not been considered; and, as this is somewhat more in the cantilever type than in the tension-ring type, correction of this omission would be in favor of the tension-ring type. On the other hand, in the cantilever-wall type it is possible and economical to make the thickness of the wall greater than assumed in the foregoing equations, so that the percentage of metal is less than 3. The addition of this condition will favor the cantilever-wall type.

Applying this relation to Mr. Flad's executed reservoir, with the bottom of the wall at Elevation 97 and high-water line at Elevation 128, and a depth of 31 ft., it appears that the maximum economical diameter would be 134 ft. Mr. Flad's tank is somewhat larger than this, with a diameter of 150 ft.; but there are some conditions stated in his paper which amply explain and justify his choice of design. The foundation is stated to be river silt, and is no doubt ill adapted to support cantilever retaining walls satisfactorily. It is obvious that the more uniform loading given by the tension-ring type of tank justifies the use of additional steel reinforcement.

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PAPERS AND DISCUSSIONS

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TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.

Discussion.*

By Leonard S. Smith, M. Am. Soc. C. E.

Mr. Smith.

Leonard S. Smith, M. Am. Soc. C. E. (by letter).—This paper is a notable contribution to the literature of topographic surveying, and is of special interest to the writer because of its comparisons with the topographical work of the Barlow, United States and Mexican Boundary Survey.† In 1892-93 the writer made most of the actual stadia measurements on the tangent or boundary line from El Paso to Yuma, Ariz., besides sharing in the topographic traverses in the same region. In the 20 years which have elapsed since this work was done, he has given a good deal of thought to this subject in connection with both his instruction in topographic surveying at the University of Wisconsin, and also in certain stadia surveys of Wisconsin's most important lakes and rivers, some of which are described later. This work included the running of more than 2 000 miles of stadia traverses, and was secured under a great variety of climatic and other conditions, and for a variety of purposes. This experience has often illustrated the fact that there are many ways in which to make stadia measurements (as well as steel tape measurements), all depending on the character of the work and the accuracy required. Stadia measurements, in fact, should mean many things to all men, not one thing to some men.

* Continued from January, 1914, *Proceedings*.

† "Topography on the Survey of the Mexico-United States Boundary," by J. L. Van Ornum, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 259.

Mr. Smith.

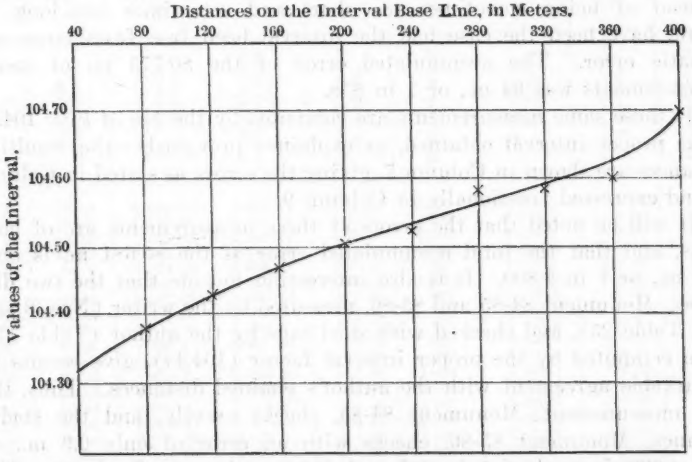
The paper includes certain criticisms of the Barlow International Boundary work. The reader must not forget that this field work, as a matter of fact, was completed more than 20 years ago. In what line of engineering endeavor would it not be possible to criticize the work of two decades ago, in the light of present knowledge? It would be difficult to show that this work was not the most wisely planned topographic work of its day. The country covered included hundreds of miles of trackless deserts interspersed with scores of mountain ranges, all uninhabited and in most part devoid of water. In the Rio Grande boundary work, such extremes were not present, and, in general, conditions were much more favorable for accurate work.

The writer agrees thoroughly with Mr. Follett that adjustable stadia wires are "a delusion and a snare", also that rod levels for keeping the rod plumb are generally unnecessary, but he cannot agree with his statements regarding the general needlessness of an interval determination. It would be a very unimportant piece of work indeed on which the writer would use a steel tape without testing its length as reported by the dealer. The testing of the wire interval of a transit supplies the same information as the testing of the steel tape. So far as concerns the need of either test, it must be admitted that the wire interval is much more likely to change in the field than is the length of a steel tape. The author advocates sending the transit from the field to the manufacturer for readjustment of the interval whenever changes make this necessary.

The writer knows of engineers who send their transits to the instrument maker for even the ordinary adjustment, but he does not favor such a practice. When the adjustments and calibrations of a transit are made in the field by the engineer who uses such instrument, better results may generally be expected than if such adjustments and calibrations are made by some unknown mechanician in a distant city. Moreover, the danger of changes during transportation is always to be considered.

A careful examination of the fifty transits belonging to the Surveying Department of the University of Wisconsin, as well as a score of other instruments with which the writer at some time has been familiar, discloses the fact that only rarely has the stadia interval been found to be even approximately 100. In fact, even if the stadia wires were exactly one-one hundredth part of the focal length of the lens in the factory, one could not be certain that they would intercept 1 ft. on the rod for every 100 ft. of distance when used under vastly different circumstances in the field. This is not a matter of theory with the writer, but of frequent and accurate determination. He feels more certain to-day than he did 20 years ago that the general rule regarding the calibration of instruments applies equally well to the stadia interval; namely, that such calibrations, as far as con-

venient, should be made under the same circumstances as those to be met in the field work. Mr. Follett does not seem to think such precautions likely to result in more accurate work, and quotes certain stadia measurement made by the writer in 1892 to prove his point. It is now proposed to use these same stadia measurements, together with seventeen others, all checked by triangulation and steel tape measurements, to show that had not less but more care been taken in one important particular, a much higher accuracy would have resulted. Mr. Smith.



CHARACTERISTIC INTERVAL CURVE, U.S.-MEXICAN BOUNDARY LINE, 1892-93.
FIG. 4.

In determining the stadia interval the method was to read the rods every 40 m. on a measured base line 400 m. long. A single determination of the interval was thus secured at each 40-m. point. These values are platted as the ordinates and the distances as abscissas of a curve, which, much generalized, is shown in Fig. 4. The adopted or working value of the interval was the average of all the ten values obtained at the various base distances. On account of the systematic differences in these partial intervals, this method would be correct only in case the stadia readings of the later field measurements were distributed approximately over the same range in distance as those of the interval determination. Now, in measuring the boundary line, rod readings at greater distances than 270 m. were not taken. The writer has reviewed the original field books concerned, and has found that had the computations for the stadia interval been restricted to the corresponding distance limit (270 m.), the resulting working interval would have been 104.44 instead of 104.60. This method was not followed at the time the measurements were made because its necessity then was not realized.

Mr. Smith.

When preparing their report, 2 years later (1895), this feature of determining an interval was discussed and approved by the U. S. Commissioners. Table 25 is a record, which was checked by triangulation or steel tape measurements, of the writer's boundary stadia measurements and taken from page 235 of the U. S. Commissioner's report. A study of this table discloses the fact that the sign of the error of every stadia measurement checked is positive, that is, the distances, as measured by the stadia, were in every case too long, instead of being sometimes too short and sometimes too long, as would have been the case had the interval been free from large systematic error. The accumulated error of the 80 775 m. of stadia measurements was 94 m., or 1 in 858.

If these same measurements are computed by the use of $k = 104.44$ —the proper interval obtained, as explained previously—the resulting distances are shown in Column 7, giving the errors as stated in Column 8, and expressed fractionally in Column 9.

It will be noted that the errors of these measurements are of both signs, and that the total accumulated error of the 80 681 m. is only 28.8 m., or 1 in 2 800. It is also interesting to note that the two distances, Monument 84-85 and 85-86, measured by the writer (Nos. 8 and 9 of Table 25), and checked with steel tape by the author (Table 17), when computed by the proper interval factor (104.44), give results in remarkable agreement with the author's chained distances. Thus, the first measurement, Monument 84-85, checks exactly, and the stadia distance, Monument 85-86, checks with an error of only 0.9 m., or 1 in 4 930, instead of 1 in 657 and 756, as given in Table 17. The fact that the interval factor, 104.44, was computed from the actual field notes and published 15 years before the author's steel-tape measurements were made, should prove the entire reliability and independence of the writer's statements and comparisons. This work emphasizes the fact that an accurate stadia interval must be secured in order to prevent large systematic errors. The greatest advantage of stadia measurements—the certain approximate balancing of the accidental errors—is neutralized if large systematic errors are allowed to enter.

The measurements given in Table 25 are less than 10% of the boundary measurements made by the writer, and it was the general belief that they constituted the poorest work of the 800 000 m. In fact, they were checked over with the tape because of the large discrepancies from the Mexican measurements.

The writer shares the opinion, expressed in Mr. W. N. Brown's discussion, that the author under-estimates the accuracy with which stadia measurements may be made. Moreover, this greater accuracy can be secured without the expense of much labor or time. The office work of preparing an interval table* does not require more than an

* Johnson and Smith's "Surveying," pp. 76-78.

hour, and with such a table the day's field notes can be reduced in 15 min. The time taken for an interval determination will be a small part of that involved in sending the transit to the instrument maker for adjustment of the wires. Mr. Smith.

TABLE 25.—ACCURACY OF STADIA MEASUREMENTS ON BARLOW, UNITED STATES–MEXICO BOUNDARY LINE, 1892–94.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
No.	Between monuments.	$k = 104.60$ U. S. stadia, in meters.	Triangulation and steel tape, in meters.	Error of stadia, in meters.	Ratio of error.	$k = 104.44$ U. S. stadia.	Error of stadia, in meters.	Ratio of error, 1 in.
1	43–44	3 691.74	3 690.14	+1.60	2 306	3 686.1	−4.04	913
2	46–47	4 779.95	4 773.91	+6.04	790	4 772.7	−1.21	3 944
3	48–49	4 445.93	4 443.17	+2.76	1 610	4 439.0	−4.17	1 064
4	62–63	3 340.28	3 334.02	+6.26	532	3 335.2	+1.18	2 820
5	69–70	3 332.65	3 324.18	+8.47	392	3 327.5	+3.32	1 000
6	78–79	5 992.19	5 987.53	+4.66	1 280	5 983.0	−4.53	1 320
7	83–84	6 067.67	6 060.42	+7.25	830	6 058.4	−2.02	3 020
*8	84–85	3 983.66	3 977.6	+6.06	660	3 977.6	0.0	Infinity.
*9	85–86	4 449.98	4 444.1	+5.88	755	4 443.2	−0.9	4 930
10	86–87	6 332.4	6 325.51	+6.89	920	6 322.9	−2.61	2 420
11	87–88	6 698.31	6 685.18	+13.13	509	6 688.2	+3.0	2 230
12	91–92	5 698.71	5 693.35	+5.36	1 060	5 690.1	−3.25	1 750
*13	92–93	4 465.2	4 462.58	+2.62	1 700	4 458.4	−4.18	1 060
14	93–94	3 310.4	3 308.87	+1.53	2 160	3 305.4	−3.47	924
15	94–95	3 478.41	3 475.07	+3.34	1 040	3 473.2	−1.90	1 820
16	95–96	3 074.85	3 073.61	+1.21	2 540	3 070.2	−3.41	900
17	96–97	3 627.78	3 623.94	+3.84	943	3 622.3	−1.64	2 200
18	107–108	1 530.27	1 527.33	+2.94	510	1 528.0	+0.67	2 280
19	119–120	2 475.42	2 471.31	+4.11	600	2 471.7	+0.39	6 330
	Totals.	80 775.80	80 681.82	+94.0	858	80 653.1	−28.8	2 800

* Stadia measurements referred to by the author in Table 17.

Greater accuracy in the stadia measurements may involve the saving of expensive control.

This is well illustrated by the methods adopted by the Wisconsin Water-Power Survey, a co-operative survey between the State and the U. S. Geological Survey.*

The specifications for this work were made nominally by the U. S. Geological Survey, but the writer was permitted to make several important changes which either added to the value of the work or greatly reduced its cost. One modification in the field work was the substitution of the magnetic needle for the control of all directions, the magnetic declination being frequently determined from Polaris observations. All distances were read by stadia and water levels; shore lines and contours were determined either by level or vertical-

* This work was done in 1906–07.

Mr. Smith.

angle readings. Only every other turning point was occupied by the transit, a method which increased the speed by about 33 per cent.

Such horizontal control as was needed for the purposes of a preliminary survey was secured by checking on the section lines where they crossed the river, a feature which had the additional advantage of showing also the ownership of the riparian rights.

The vertical control was secured by running Wye-level lines and establishing permanent bench-marks on or near the river at intervals of about 2 miles, 400 miles of such levels being run at a total expense of $1 277. The unit cost of these levels varied from $2.16 per mile in a settled country with good roads to $4.85 per mile in a region devoid of roads and settlements. The instrument men on this work were junior civil engineering students at the University of Wisconsin.

As little on the subject of the accuracy of transit leveling has appeared in engineering literature, it may be of interest to examine in detail the accuracy of this work, which is shown in Tables 26, 27, and 28. In judging its accuracy, it should be remembered that the average turning-point distance was about 1 500 ft., and that the young men had had very little instrumental experience. The sensibility of the level corresponded to $\frac{1}{10}$ in. = 15″ of arc. The transits had object glasses $1\frac{1}{4}$ in. in diameter and the eye-piece magnified 24 times.

TABLE 26.—ACCURACY OF TRANSIT LEVELING ON BLACK RIVER, WISCONSIN.

No. of check.	Distance, in miles.	Error in Stretch.				Accumulative Error.		Total.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	Miles.
1	2.3	0.09	.....	+0.09	0.04	+0.09	0.04	2.3
2	2.3	0.44	.....	+0.44	0.20	+0.53	0.12	4.6
3	2.4	0.17	0.69	−0.52	0.20	+0.01	0.001	7.0
4	3.6	0.23	0.54	−0.31	0.09	−0.30	0.029	10.6
5	4.6	.....	0.14	−0.14	0.03	−0.44	0.030	15.2
6	5.2	1.50	.....	+1.50	0.29	+1.06	0.050	20.4
7	3.8	.....	0.57	−0.57	0.15	−0.49	0.020	24.2
8	3.2	1.01	.....	+1.01	0.31	−1.50	0.055	27.4
9	3.6	0.73	0.90	−0.18	0.05	−1.32	0.042	31.0
10	3.6	0.20	.....	+1.20	0.33	+2.52	0.073	34.6
11	6.3	0.49	0.59	−0.10	0.02	+2.42	0.06	40.9
12	4.8	0.31	0.75	−0.44	0.09	+1.98	0.044	45.7
13	5.2	0.27	0.17	+0.10	0.02	+2.08	0.041	50.9
14	1.8	.....	0.78	−0.78	0.40	+1.30	0.025	52.7
15	5.3	.....	0.79	−0.79	0.14	+0.51	0.009	58.0
					Average.		0.15	

Table 26 shows that the largest error per mile on any stretch of the Black River Survey was 0.33 ft., and that the average error per mile was 0.15 ft. These errors, however, compensated to a remarkable extent, so that at the end of 10, 20, 31, 41, 51, and 58 miles the corresponding accumulative errors were only 0.03, 0.05, 0.04, 0.06, 0.04.

and 0.009 ft. per mile. The same engineer, Mr. Victor Reineking, next surveyed the Flambeau River with even better results, as will be seen from Table 27. Mr. Smith.

TABLE 27.—ACCURACY OF TRANSIT LEVELING ON FLAMBEAU RIVER.

No. of check.	Distance, in miles.	Error in Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error	Per mile.	Total.	Per mile.	
1	3.4	0.27	0.05	+0.22	0.06	+0.22	0.06	3.4
2	2.5	.....	0.58	−0.58	0.23	−0.36	0.06	5·9
3	2.8	0.17	.....	+0.17	0.07	−0.19	0.02	8.7
4	2.5	.....	0.28	−0.28	0.11	−0.47	0.042	11.2
5	1.5	.....	0.08	−0.08	0.05	−0.45	0.045	12.7
6	2.2	.....	0.10	−0.10	0.045	−0.65	0.043	14.9
7	4.1	.....	0.69	−0.69	0.16	−0.134	0.070	19.0
8	4.4	0.62	.....	+0.62	0.14	−0.72	0.031	23.4
9	2.7	0.40	.....	+0.40	0.14	−0.32	0.012	26.1
10	1.3	0.20	.....	+0.20	0.16	−0.12	0.004	27.4
11	1.6	0.22	.....	+0.22	0.13	+0.10	0.003	29.0
12	1.9	0.38	.....	+0.38	0.20	+0.48	0.015	30.9
13	5.0	.....	0.28	−0.28	0.05	+0.20	0.006	35.9
14	2.5	.....	0.68	−0.68	0.15	−0.48	0.013	38.4
15	1.2	0.34	.....	+0.34	0.22	−0.14	0.004	39.6
16	2.5	0.22	.....	+0.22	0.08	+0.08	0.002	42.1
17	2.4	0.02	.....	+0.02	0.008	+0.10	0.001	44.5
18	3.1	.....	0.27	−0.27	0.087	−0.17	0.004	47.6
19	3.1	0.09	.....	+0.09	0.028	−0.08	0.002	50.7
20	4.2	.....	0.02	−0.02	0.004	−0.10	0.002	54.9
21	1.0	.....	0.06	−0.06	0.06	−0.16	0.006	55.9
22	1.8	.....	0.27	−0.27	0.15	−0.43	0.008	57.7
23	3.6	0.51	.....	+0.51	0.07	+0.08	0.001	61.3
24	6.6	.....	0.03	−0.03	0.005	−0.05	0.001	67.9
25	4.8	0.46	.....	+0.46	0.096	+0.51	0.007	72.7
26	1.4	0.11	.....	+0.11	0.078	+0.62	0.008	74.1
27	4.0	0.05	.....	+0.05	0.012	+0.67	0.008	78.1
28	3.2	0.22	.....	+0.22	0.070	+0.89	0.011	81.3
29	3.6	.....	0.56	−0.56	0.153	+0.33	0.004	84.9
30	3.0	0.04	.....	−0.04	0.012	+0.29	0.003	87.9
31	4.4	.....	0.03	−0.03	0.007	+0.26	0.003	92.3

TABLE 28.—ACCURACY OF TRANSIT LEVELING ON WISCONSIN RIVER.

No. of check.	Distance, in miles.	Error of Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	18.1	1.14	1.77	−0.63	0.039	−0.63	0.039	18.1
2	14.6	.....	2.73	−2.73	0.187	−3.36	0.103	32.7
3	15.0	0.45	.....	+0.45	0.03	−2.91	0.061	47.7
4	15.0	.....	0.28	−0.28	0.02	−3.19	0.050	62.7
5	27.0	0.45	0.55	−0.10	0.004	−3.29	0.036	89.7
6	8.0	0.04	0.01	+0.03	0.003	−3.26	0.033	97.7
7	16.3	0.16	0.53	−0.37	0.023	−3.63	0.032	114.0
8	6.3	0.10	0.04	+0.06	0.01	−3.57	0.029	120.3
9	25.0	0.86	.....	+0.86	0.03	−2.71	0.019	145.3
10	23.0	0.85	0.17	+0.68	0.02	−2.03	0.012	168.3
11	10.7	0.49	0.16	+0.33	0.03	−1.70	0.010	179.0
12	6.0	0.54	.....	+0.54	0.09	−1.16	0.006	185.0
13	6.0	0.38	.....	+0.38	0.06	−0.78	0.004	191.0
14	9.0	0.34	0.13	+0.21	0.02	−0.57	0.003	200.0

Mr. Smith.

TABLE 29.—ACCURACY OF TRANSIT LEVELING ON PESHTIGO RIVER.

No. of check.	Distance, in miles.	Error of Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	2.1	0.16	.....	+0.16	0.08	+0.16	0.08	2.1
2	1.9	.....	0.17	+0.01	0.09	+0.01	0.002	4.0
3	3.6	0.72	.....	+0.72	0.19	+0.71	0.09	7.6
4	1.5	0.13	.....	+0.13	0.09	+0.84	0.09	9.1
5	4.7	0.32	.....	+0.32	0.07	+1.16	0.08	13.8
6	2.5	0.13	.....	+0.13	0.05	+1.29	0.078	16.3
7	5.0	0.02	.....	+0.02	0.004	+1.31	0.062	21.3
8	5.0	0.16	.....	+0.16	0.032	+1.47	0.056	26.3
9	7.6	0.17	.....	+0.17	0.022	+1.64	0.049	33.9
10	4.0	0.0	0.94	−0.94	0.23	+0.70	0.018	37.9
11	5.1	.....	0.46	−0.46	0.09	+0.24	0.005	43.0
12	2.4	.....	0.04	−0.04	0.017	+0.20	0.004	45.4
13	3.5	0.81	.....	+0.81	0.23	+1.01	0.02	48.9
14	1.3	.....	0.08	−0.08	0.02	+0.98	0.02	50.2

The largest error was 0.22 ft. per mile, the average being 0.09 ft. per mile. Of the 31 checks, 14 show minus and 17 plus errors. This compensation, as shown in Table 27, is truly surprising. The total accumulative error at the end of the 92.3 miles was only 0.26 ft. or 0.027 ft. per mile.

Table 28 gives the results of transit leveling on the Wisconsin River Survey. Because of a misunderstanding of directions, the leveling on the first 32.7 miles of river was done by reading vertical angles, with the result that the largest errors are found in this stretch. In the remaining 168 miles of levels, the average error per mile is 0.04 ft., and the total accumulative error on the 200 miles is 0.57 ft.

TABLE 30.—SUMMARY OF TOTAL AND UNIT COSTS OF CO-

River surveyed.	Spirit Levels.								Topography.					
	Total miles.	Average per work day, in miles.	Lost time.		Miscellaneous.		Gross cost.		Total miles.	Average per work day.	Lost time.		Miscellaneous.	
			Days.	$	Days.	$	Total.	Per mile.			Days.	$	Days.	$
Wisconsin.....	1 873	4.9	4	35	4	35	403	2.16	197.0	2.5	16	142	15	135
Eau Claire.....	.....	.....	.....	.....	.....	.....	.....	.....	25.8	3.2	10	88	3	18
Peshtigo.....	40	5.0	7	59	5	38	194	4.87	81.7	3.3	1	12	13	182
Black.....	69	3.9	4	36	1	9	207	3.00	63.0	3.0	6	50	1	8
Flambeau.....	102	3.9	9	99	5	55	467	4.58	119.5	3.0	15	165	9	99
Totals and averages.	3 983	4.5	24	229	15	137	1 271	3.19	487.0	2.86	48	457	41	442

NOTE:—"Lost time" includes Sundays, holidays, and stormy weather. Miscellaneous

Mr. Smith.

The 82 miles of transit levels run on the Peshtigo River were, for the most part, in rough, rocky, and heavily timbered country. The largest error per mile in any stretch was 0.23 ft., the smallest was 0.02 ft., with an average of 0.09 ft. The largest accumulated error was 1.64 ft., at a point 33 miles from the beginning. The accumulative error per mile gradually decreased from 0.09 ft. near the beginning to only 0.02 ft. at the end of the 82 miles.

It is perfectly manifest, from this record of 400 miles of transit levels, that for general preliminary surveys transit levels are amply sufficient without Wye-level control. Such omission would have reduced the cost of the survey by an average of $3.24 per mile.

Very careful records of the most important elements of cost data were kept, and are given in Tables 30 and 31. These elements, together with the costs of supervision, are assembled in Table 31. The greater cost of the Flambeau River work was due to working in the winter on the ice. The winter happened to be one of unusual severity, with very low temperatures and many deep snows. This resulted in the loss of about 30 work days, which increased the cost by $3.50 per mile of survey.

The surveys were mapped on a scale of 1 000 ft. to 1 in., and published by the U. S. Geological Survey on a scale of 2 000 ft. to 1 in. It will be noted that the total cost of 487 miles of river surveys, including both field and office expenses, was $4 798.30, or only $9.85 per mile for both profile and plan with land sections. This work furnishes a good example of the adaptability of the transit and stadia method.

OPERATIVE WATER-POWER SURVEY IN WISCONSIN, 1905-06.

Gross cost.		Mapping. Net cost.		Gross cost.		Total per mile.		Time not Spent in Field Work. Lost time.		Miscellaneous.		Σ Time.		Σ Cost of field and office work.	
Total.	Per mile.	Map.	Profile.	Map.	Profile.	Map.	Profile.	Days.	$	Days.	$	Days.	$		
981	4.97	120	11	160	16	0.81	0.08	22	182	25	210	47	392	$1 560	Dugan.
177	6.87	9	2	9	2	0.36	0.06	10	88	3	18	13	106	188	"
521	6.38	32	3	37	3	0.46	0.04	10	76	21	221	31	297	755	"
248	3.94	....	.....	51	6	0.80	0.10	10	90	2	18	12	108	511	Reineking.
768	6.43	....	.....	94	12	0.80	0.10	24	264	14	54	38	318	1 341	"
2 695	5.53	....	.....	351	39	0.72	0.08	76	700	65	521	141	1 121	$4 355	

time includes traveling and all causes not included in "lost time."

Mr. Smith.

TABLE 31.—TOTAL AND UNIT COST OF WISCONSIN CO-OPERATIVE RIVER SURVEY, 1905-06.

L. S. SMITH, Hydrographer in Charge.

River surveyed.	SPIRIT LEVELING.			TOPOGRAPHY.			MAP AND PROFILES.			Cost of reconnaissance and supervision, per mile.	Total cost per mile of river.	Total cost.
	Total miles.	Total cost.	Cost per mile.	Total miles.	Total cost.	Cost per mile.	Total miles mapped.	Total cost.	Cost per mile.			
Wisconsin..	187.3	$403	$2.16	197.0	$981	$4.97	197.0	$171	$0.89	$0.73	$8.61	$1 698.00
Eau Claire.	0.0	0	0.00	25.8	177	6.87	25.8	11	0.44	1.00	8.29	213.80
Peshtigo....	40.0	194	4.85	81.7	521	6.36	81.7	40	0.50	0.70	9.98	815.20
Black.....	69.3	213	3.10	63.0	213	3.38	63.0	57	0.90	1.10	8.76	552.30
Flambeau..	102.0	467	4.58	119.5	796	6.69	119.5	106	0.90	1.25	12.71	1 519.00
Total.....	398.6	$1 277	$3.24	487.0	$2 688	$5.52	487.0	$385	$0.79	$0.90	$9.85	$4 798.30

Method—Transit and stadia with Wye-level control.
Scale—1 in. = 1 000 ft.
Contour interval = 10 ft. on land and 1 ft. on water.
Section lines located frequently.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

Sir WILLIAM HENRY WHITE, Hon. M. Am. Soc. C. E.*

DIED FEBRUARY 27TH, 1913.

William Henry White, the youngest child of Richard and Jane (Matthews) White, was born at Devonport, England, on February 2d, 1845. He attended the local schools until March, 1859, when, at the age of fourteen, he was apprenticed at the Royal Naval Dockyard. In addition to his practical work, he attended the Dockyard School, maintained by the Admiralty as a part of the technical training of its apprentices, where he showed his ability by winning an Admiralty Scholarship in 1863.

It was during this time that propulsion by steam was being substituted for sails, and sailing vessels were being reconstructed as screw-propellers. The use of iron in place of wood for the hulls of vessels was also being introduced, and young White had the advantage of combining practical work with the preparation of designs, the results of which were clearly shown in his highly responsible work of later years.

In 1864, the Admiralty, at the urgent request of the Institution of Naval Architects, established the Royal School of Naval Architecture at South Kensington, in connection with the Science and Art Department. This school, which was instituted in order that students in naval architecture might take advanced courses in that science, was afterward merged into the Royal Naval College at Greenwich. Sir William was one of the first students received at this newly founded college, taking first place at his entrance examinations in 1864, which standing he maintained during his three years of attendance. He was graduated in 1867, with the highest honors, as a Fellow (first-class). In 1870 he was made Professor of Naval Architecture at this school, which position he held until 1881. This work was in addition to his duties at the Admiralty. Among his pupils during this period were men who afterward became chief constructors and naval architects in the various navies of the world, and it was largely due to his influence that the school's high standing was acquired and maintained.

Immediately after his graduation from the Royal School of Naval Architecture, Sir William entered the Admiralty as Private Secretary to Sir Edward Reed, Chief Constructor, being engaged in the solution of scientific problems in naval architecture, etc. It was while he oc-

* Memoir prepared by the Secretary from information on file at the Society House.

cupied this position that he was brought directly in touch with the design of armored vessels which were then a new invention, thus adding to his experience as a designer and constructor.

In 1870, on the resignation of Sir Edward Reed and the appointment of a Commission to continue his work, of which Sir Nathaniel Barnaby was Temporary President, Sir William was made Professional Secretary to the Commission. In this position, and with the aid of his life-long friend, the late Mr. William John, he had charge of, and carried out, numerous experimental inquiries in regard to the stability of ships for the Commission appointed to investigate the capsizing of the ironclad *Captain*, in the Bay of Biscay. The results of these inquiries were embodied in a paper* on the subject presented before the Institution of Naval Architects, which greatly advanced the science of ship design.

In 1875, Sir William was promoted to the rank of Assistant Constructor, and, in 1881, he was advanced to that of Chief Constructor. While in this position he effected the organization of all the trained architects in the Admiralty into one corps—the Royal Corps of Naval Constructors—which organization has proved of great service to the British Navy. In 1883, when Sir William Armstrong was establishing his great shipyard at Elswick, England, he offered to Sir William the difficult feat of organizing and directing the Warship Building Department. While in this position he made designs for naval vessels for Austria, Italy, Japan, China, and Spain, and had charge of the construction of several vessels for the British Navy, among which was the *Victoria* which, in 1893, was sunk in the Mediterranean by a collision, with great loss of life.

In 1885, Sir Nathaniel Barnaby resigned his position at the Admiralty on account of ill-health, and on his recommendation, Sir William, having been released from his contract with Sir William Armstrong, Mitchell and Company, was appointed Director of Naval Construction and Assistant Controller of the Navy, which position he held until February, 1902, when he also was compelled to resign on account of ill-health.

Sir William began his work as Director of Naval Construction at the Admiralty at the beginning of a period of expansion in British Naval affairs. Prior to that time no uniformity in the vessels of the fleet had been attempted, and the reorganization of the administrative and operative departments was demanded. The Northbrooke programme, which comprised the carrying out of the construction of a number of new ships and the rapid completion of those already begun, was his first task, and the work was accomplished so expeditiously that when the Naval Defence Act of 1889 was passed, providing for

* "The Calculation of the Stability of Ships, and Some Matters of Interest Connected Therewith," by W. H. White and W. John, *Transactions*, Inst. of Naval Archts., Vol. XII, p. 77.

the construction of 70 ships at a cost of £22 000 000, sterling, Sir William had a chance to carry out his idea of homogeneity in a fleet, namely, ships bearing a distinct relation to each other and to the fleet as a whole. The Spencer programme of 1894 and the Goschen programme of 1896, the latter comprising the construction of the first dreadnought of the British Navy, were also carried out under his supervision. The development of the torpedo boat and the torpedo-boat destroyer was due to his skill, and he also designed the river gunboats built for the Nile Expedition under Lord Kitchener. When he retired in 1902, he had had responsible charge of the design and construction of 245 vessels, valued at about £100 000 000, sterling. All this vast work was done under constantly changing conditions of material, type, size, speed, armament, etc., and to him may be attributed the introduction of several innovations in British warships, notably that of water-tube boilers and the use of oil fuel for firing boilers. He had entered the Naval Service before the first real ironclad was ordered, and had seen the Navy develop from one of wooden ships to the present-day steel dreadnought.

He was awarded a C. B. in 1891 and a K. C. B. in 1895, and on his retirement from the Admiralty in 1902, Parliament voted him a special grant for "exceptional services to the Navy."

After a rest and the recovery of his health, Sir William began practice as a Consulting Naval Architect, and was engaged on many important works. He was a member of the Cunard Commission which decided the type of machinery for the *Lusitania* and the *Mauretania;* a Director of the firm of Swan, Hunter, and Wigham Richardson, Limited, the builders of the *Mauretania,* during her construction; a Director of the Parsons Marine Turbine Company; and a Director of the Grand Trunk Railway Company after that Company became the owner of steamships. He designed steamers with geared turbines for service between India and Ceylon, and was a Member of the Government Commission to investigate the question of load lines of merchant ships.

On February 27th, 1913, Sir William suffered a paralytic stroke at his offices in Westminster, and died the same day at Westminster Hospital to which he had been removed.

He was twice married, his first wife having been Miss Alice Martin who died in 1886. In 1890, he was married to Miss Annie Marshall who, with three sons, all of whom are officers in the British Navy, and one daughter, survives him.

Sir William was a frequent contributor to technical and engineering journals and to the publications of technical and scientific societies. He was also the author of several books, his "Manual of Naval Architecture" and "Treatise on Shipbuilding" having become classics, the former being translated into German, Italian, Russian, and Spanish.

His most notable contributions were made to the *Transactions* of the Institution of Naval Architects, the best known being the papers on the stability of ships (already referred to), the rolling of sailing ships, and the effect of bilge keels on rolling, and his description of the design of the battleship *Royal Sovereign*. That his versatility was not confined to engineering, but extended over a wide range of subjects, is shown by his contributions to various newspapers and magazines.

He was always greatly interested in questions of engineering and technical education, owing, perhaps, to the difficulties he experienced in acquiring his own training. He was Chairman of the Committee on the Education and Training of Engineers, appointed by the Institution of Civil Engineers in 1903, and was also a member of the Governing Body of the Imperial College of Science and Technology.

Sir William was connected with many engineering and scientific societies and associations, to which he was always ready to give advice and assistance, and many of these had honored him with offices of distinction. He was a Fellow of the Royal Society; Honorary Vice-President of the Institution of Naval Architects; Past-President of the Institutions of Civil Engineers, Mechanical Engineers, Marine Engineers, Junior Engineers, and the Institute of Metals; President-Elect of the British Association for the Advancement of Science, having been President of the Mechanical Science Section. He was also an Honorary Member of many other British and foreign technical societies, including the American Society of Mechanical Engineers and the Society of Naval Architects and Marine Engineers.

In 1911 he was awarded the John Fritz Medal by a Board of Award appointed by the four leading American engineering societies—Civil, Mining, Mechanical and Electrical—for "notable achievements in naval architecture." He had also received honorary degrees from many colleges and universities, among which were LL.D. from Glasgow, D. Sc. from Cambridge, Durham, and Columbia (New York City) Universities, and D. Eng. from Sheffield. He also belonged to the Athenæum and British Empire Clubs.

In regard to his personal characteristics, the following* was written by one who had been on terms of close friendship with him for more than 30 years:

"He was beyond compare the straightest and most generous man alive. He was, without exception, the most remarkable man I have known, and as honest as the day. From the start he had never a soul to help him, but achieved distinction by his own ability and industry. A foreign government made overtures to him to superintend the total reconstruction of its then somewhat infirm war flotilla. White, with that fine sense of honour which was inseparable from the man, re-

The Times (London), February 28th, 1913.

fused a most lucrative offer because he considered the value he could return for the great monetary payment held out to him, could only have been acquired by his intimate knowledge gained in the service of our Admiralty. The amount of work that he did without repayment in those educational institutions which are bringing forward the most intelligent men of the new generation, was beyond compare. His self-negation, accurate knowledge, and charming manner extraordinarily enhanced by a great gift of description, made him such a professor of scientific and applied mechanics as the world has seldom seen."

Sir William Henry White was elected an Honorary Member of the American Society of Civil Engineers on December 16th, 1904.

HORACE GUY MERRICK, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 30TH, 1913.

Horace Guy Merrick was born at Libertyville, Ill., on January 29th, 1879. His family moved to Manistee, Mich., when he was quite young.

Mr. Merrick's education began in the public schools at Manistee, and he was graduated from the High School of that place in 1898. During the next three years he was engaged on various public works in Michigan. In 1901 he entered the University of Michigan, from which institution he was graduated in 1905, with the degree of B. S. in Civil Engineering.

After his graduation he was employed by the United States Government as Junior Engineer on the survey of the Great Lakes until May, 1907, when he was transferred to the work of improving the Upper Mississippi River, with headquarters at La Crosse, Wis. He remained on this work until his death.

In 1913, Mr. Merrick took the examination before the United States Civil Service Board for registration and promotion to Assistant Engineer, and passed with much credit.

He was an energetic and capable civil engineer, and was held in high esteem by his superior officers. He felt no wish to become conspicuous, professionally or otherwise, shrinking from notice rather than courting it. He always under-estimated his own ability, and worked zealously, intelligently, and successfully from love of his Profession.

Mr. Merrick was elected an Associate Member of the American Society of Civil Engineers on May 7th, 1913. He was also a member of the Wisconsin Society of Civil Engineers.

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* Memoir prepared by W. A. Thompson, M. Am. Soc. C. E.

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PAPERS AND DISCUSSIONS

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THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

By F. Lavis, M. Am. Soc. C. E.

To be Presented April 1st, 1914.

Synopsis.

The question of the gauge of railways, though not important in the United States, where it was solved many years ago by the adoption of 4 ft. 8½ in. as a standard, is a live issue in South and Central America, where there are lines of nine different gauges, those most in use being 1 meter, 4 ft. 8½ in., and 5 ft. 6 in. During a recent visit to the Argentine the writer made an investigation of this subject, some of the results of which are embodied in this paper.

The railways of South America have been most largely developed, thus far, by European capital and engineers, but, owing to the changed conditions which have placed, or will soon place, the United States in the position of an importer of food supplies and a seeker for markets for its manufactured products, it seems evident that we must take a much greater and more intimate interest in South and Central American affairs. This question of gauge, therefore, is one which may interest many American engineers in the not far distant future.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The subject is treated in the order and under the several different headings mentioned below, the conclusions being briefly summarized as follows:

Curvature.—Within the limits of the practical operation of railroads, any radius curvature which is feasible on meter gauge is as practical on 4 ft. 8½-in. or even 5 ft. 6-in.

Gradients.—The working of trains on steep gradients is not affected by gauge, except in so far as the narrow-gauge cuts down the locomotive capacity and is, therefore, at a great disadvantage.

Location.—In view of the above, the narrow-gauge (3 ft. 0-in. to 3 ft. 6-in.) does not permit better adaptability of the alignment to the conformation of the ground.

Cost of Construction.—This is so little more for 4 ft. 8½-in. than for 3 ft. 0-in., or 3 ft. 6-in., as to be more than counterbalanced by economy in operation.

Speed.—This is an important feature in railway operation, which is adversely affected by narrow-gauge.

Stability.—The relative stability of narrow-gauge trains in motion is much less than that of trains on medium- or broad-gauge, and, consequently, high speeds are impractical on narrow-gauge, and may be dangerous.

Track Stresses.—Much higher track stresses are produced on the narrow-gauge. There is greater impact, and therefore more difficulty in maintaining track on narrow- than on broad-gauge.

Rolling Stock—Cars.—The greater width in proportion to the gauge of narrow-gauge cars is considered a necessary evil rather than an advantage, and it is shown that there is probably no advantage to be gained by making the broad (5 ft. 6-in.) gauge cars any wider than they are at present. In the matter of passenger cars, the narrow-gauge is at a decided disadvantage both as to cost per unit of accommodation and in dead weight per passenger. In freight cars there is little difference in cost within the limits of size possible on the narrow-gauge, though cars of twice the capacity are available for medium- or broad-gauge. The increase in weight of modern freight trains, in order to decrease the cost of operation has made heavier draft rigging necessary, thus tending to increase the proportion of dead to paying load to a much greater extent on cars of small capacity than on the larger ones, than has been the case heretofore.

Locomotives.—The cost per unit of capacity of locomotives is about the same for each gauge, up to the limit of the narrow-gauge, which limit for all types is about half that of the 4 ft. 8½-in. or 5 ft. 6-in. There is an economy in the use of heavier types of locomotives not available for the narrow-gauge, and the use of cars of large capacity is a necessity for cheap transportation.

Train Resistance.—The train resistance is decreased by the use of larger units.

Capacity.—Figures are given tending to show that the relative traffic capacity of narrow-gauge lines is not more than half that of medium- or broad-gauge.

Heavier Engines and Trains.—There is a general discussion showing the part played by the increase of train loads in reducing costs in the United States.

Cost of Operation.—The ton-mile costs on both Indian and Australian railways, as well as those in the Argentine, are less on the medium- and broad-gauge than on the narrow-gauge, although the train-mile costs are often higher.

The lowest ton-mile costs on any railway in the Argentine are more than twice as high as they are in the United States. The saving in cost of operation, due to the use of larger cars and heavier trains, would be considerable, and would in itself go a long way toward meeting the interest charge on the additional cost of the wider gauge.

Cost of Maintenance.—Maintenance of way probably costs more per traffic unit on narrow- than on medium- or broad-gauge on any but lines of very light traffic.

Maintenance of equipment should be less on medium- or broad-gauge by reason of the larger capacity and, consequently, lesser number of locomotives and vehicles, provided there is business to warrant the use of these larger units.

Need of Efficient Transportation Machine.—The necessity of an efficient transportation machine to develop the country properly is pointed out, as well as the part played by the railways in developing the United States, the country most nearly comparable with Brazil and the Argentine.

There are two appendices: Appendix A gives a short account of the history of the gauge question in the United States, with some notes of its present status in India, Australia, South Africa, etc. Appendix

B contains a general description of the topography, physical characteristics, and business conditions of Southern South America, and the development and present situation of its railways.

It is shown that the development and growth of this area is most nearly comparable to that of the United States, and for its exploitation a transportation machine not less efficient than that of the railways of this country is necessary. The growth of the railways of the Argentine is reviewed in some detail, showing their development within the past few years from a series of little local lines which were not much affected by the gauge to a series of fairly extensive systems, where the distances to be covered and the necessary weights of trains to be handled are showing the deficiencies of the narrow-gauge. This growth, and the increasing distances from the seaboard to new areas being opened up, all point to the need of the most efficient transportation system for its proper development.

It is believed to be shown that the narrow-gauge is an inferior transportation machine; that, as far as new lines are concerned, there is little saving and no economy in their construction, and that this gauge is inadequate for the development of areas as large as those under consideration; that the 5 ft. 6-in. gauge offers little if any advantage over the 4 ft. 8½-in.; that, taken altogether, the latter, which has been adopted as the standard in North America, Europe, Western Asia, and Australia, is best adapted to the development of this region; that new lines should be built to this gauge, and that it is economically sound to make the expenditures necessary to change the existing narrow-gauge lines to this standard.

It is pointed out that the cost of changing the gauge of the existing lines cannot be considered entirely alone, and figures are presented showing the estimated additional cost of the whole system of standard-gauge lines at the end of 20 years. The figures tend to show that, including the cost of changing the existing narrow-gauge lines, and taking into account the additional cost of building the new lines of standard- instead of narrow-gauge, that the final standard-gauge system will have cost about $5 000 per km. more than if the narrow-gauge is perpetuated, and that, under certain assumptions, such as the greater need of stone ballast, the earlier requirements of double track, etc., on the narrow-gauge, this estimate would probably be materially reduced.

Figures are given, which, taking into consideration only those items which are readily estimated in money values, tend to show that the saving in cost of operation would be sufficient to cover the interest charges on the additional cost, and, besides all this, there is the fact that the narrow-gauge is an inferior transportation machine not adapted to the development of the transportation system of a continent.

The question of the gauge of railways was settled in North America many years ago, and, so far as regards this country, or even Europe, any discussion of the matter is purely academic. The development of railway transportation in the rest of the world, however, except in certain limited sections, is as yet in its infancy, the only sections of the vast area lying to the south of the United States where any substantial progress has been made being parts of the Argentine, Southern Brazil, Chile, Peru and Mexico, and these by lines of many different gauges, varying from 2 ft. 6 in. to 5 ft. 6 in.

The writer was asked recently to make a study and report on the gauge question, as affecting the future development of the Argentine, and, as the subject is a vital one in all countries south of the United States —or perhaps more correctly south of Mexico—it is thought that the presentation of those parts of the report which are of general interest may not be untimely. This, perhaps, is especially the case now, when the United States is beginning to be forced to look for foreign markets for its manufactures, with South America as the natural field for its enterprise, and when, consequently, the interest which American engineers are likely to have in this development may be of considerable importance.

The probable future growth of the southern part of South America may be fairly comparable with that of the United States during the past 50 years. The importance of the railways as a factor in this latter has been well stated by M. Colson, an eminent French engineer,* who points out how necessary cheap, efficient railway service is in the development of new countries. It is for this reason that American railway engineers, who have been intimately connected with the railway development of their own country, should be interested in the future of South America, as their own experience should enable them to be among the best judges of the necessities of countries which, like

* See p. 614.

those in southern South America, are entering on an era of expansion comparable only to that of their own.

Europe, like the United States, long ago adopted the 4 ft. 8½-in. gauge as the standard, but the gauge question is still being actively discussed in India, where the standard is 5 ft. 6 in., with a considerable mileage 1 m. and less. Australia, where the gauges are 5 ft. 3 in., 4 ft. 8½ in., and 3 ft. 6 in., according to recent reports, has only recently decided to unify them and adopt 4 ft. 8½ in. as the standard; and Africa, unfortunately, seems to be definitely committed to what one eminent British engineer recently described as "the miserable 3 ft. 6 in. gauge."* China has adopted 4 ft. 8½ in., the Trans-Siberian Railway is 5 ft. 0 in., the South Manchurian Railway and the Korean railways are 4 ft. 8½ in., and the Japanese railways are mostly 3 ft. 6 in., with some 2 ft. 6 in., though it is stated that these latter are to be changed to 4 ft. 8½ in. as the result of their failure to meet the emergency caused by the Russian war.

It is, perhaps, true that the possible use of railways for war purposes can hardly be considered as an important economic factor in determining the basis on which they shall be designed—at least it is hardly as important in America, either North or South, as it might be in Europe or Asia—still the experience of the Japanese, as well as the English in India, tends to show the inferiority of the narrow gauge for transportation, especially in an emergency. In countries growing as rapidly as those of southern South America, this really means that it will not be long before the narrow-gauge lines will reach the same state of inadequacy to meet the natural development and growth of a new country as they have wherever they have been submitted to the emergency test of war.†

This is already apparent on some of the lines in the Argentine, and, in Brazil, the Paulista Railway Company is now reconstructing its narrow-gauge lines west of Rio Claro to 5 ft. 3 in., as it cannot handle the business offered, although the heaviest narrow-gauge equipment, both locomotives and rolling stock, is being used. Some further notes in regard to the experience of other countries with lines of various gauges are given in Appendix A.

* "The Railway Gauges of India," by Sir Frederick Robert Upton, *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV, p. 196.

† See, also, p. 580.

Some matters treated in this paper are, perhaps, not strictly technical, or may be considered somewhat elementary; but, as this subject is of interest to laymen as well as to engineers, and as the latter may often be called on to present the arguments, for or against the adoption of any particular gauge, to governments, the public, or investors, it has been thought desirable to include them all. In regard to some phases of the technical aspect of the subject, also, certain matters have been included which have long been accepted as almost axiomatic by engineers in North America, though it is the writer's experience that they are not always so accepted elsewhere.

The lengths of the lines of various gauges in the countries to the south of Mexico (which latter is practically all standard (4 ft. 8½ in.) gauge) are shown in Table 1.* Generally throughout the paper, when speaking of narrow gauge, it will be considered to include the 3 ft., 1 m. and 3 ft. 6 in. The medium or standard gauge is the United States, 4 ft. 8½ in. The broad is 5 ft. 3 in. and 5 ft. 6 in. It may be noted that the standard is 5 ft. 3 in. on the Irish Railways, and 5 ft. 6 in. on the Indian Railways.

TABLE 1.—Length, in Miles, of Railroads of Various Gauges in Countries South of Mexico.

	2 ft. 0 in.	2 ft. 6 in.	3 ft. 0 in.	Meter.	3 ft. 6 in.	4 ft. 8½ in.	5 ft. 0 in.	5 ft. 3 in.	5 ft. 6 in.
Guatemala	.....	.....	453	.....	.....	.....	.....	.....	.....
Salvador	.....	.....	100	.....	.....	.....	.....	.....	.....
Honduras	.....	.....	24	.....	.....	.....	.....	.....	.....
Nicaragua	.....	.....	.....	.....	172	.....	.....	.....	.....
Costa Rica	.....	.....	.....	.....	564	.....	.....	.....	.....
Panama	.....	.....	.....	.....	.....	.....	30	.....	.....
Colombia	.....	.....	340	63	18	.....	.....	.....	.....
Venezuela	110	.....	24	.....	199	.....	.....	.....	.....
British Guiana	.....	.....	.....	.....	.....	60	.....	.....	.....
Ecuador	.....	.....	.....	.....	278	.....	.....	.....	.....
Peru	.....	.....	262	8	15	994	.....	.....	.....
Bolivia	.....	468	54	.....	.....	.....	.....	.....	.....
Chile	.....	554	.....	2 359	279	517	.....	.....	497
Argentine	.....	.....	.....	6 181	.....	1 589	.....	.....	11 942
Paraguay	.....	.....	.....	.....	.....	232	.....	.....	.....
Uruguay	.....	.....	.....	.....	.....	1 374	.....	.....	.....
Brazil	.....	.....	.....	11 315	.....	.....	.....	286	.....
Totals	110	1 022	1 257	19 926	1 525	4 766	30	286	12 439

* Compiled from "Universal Directory of Railway Officials."

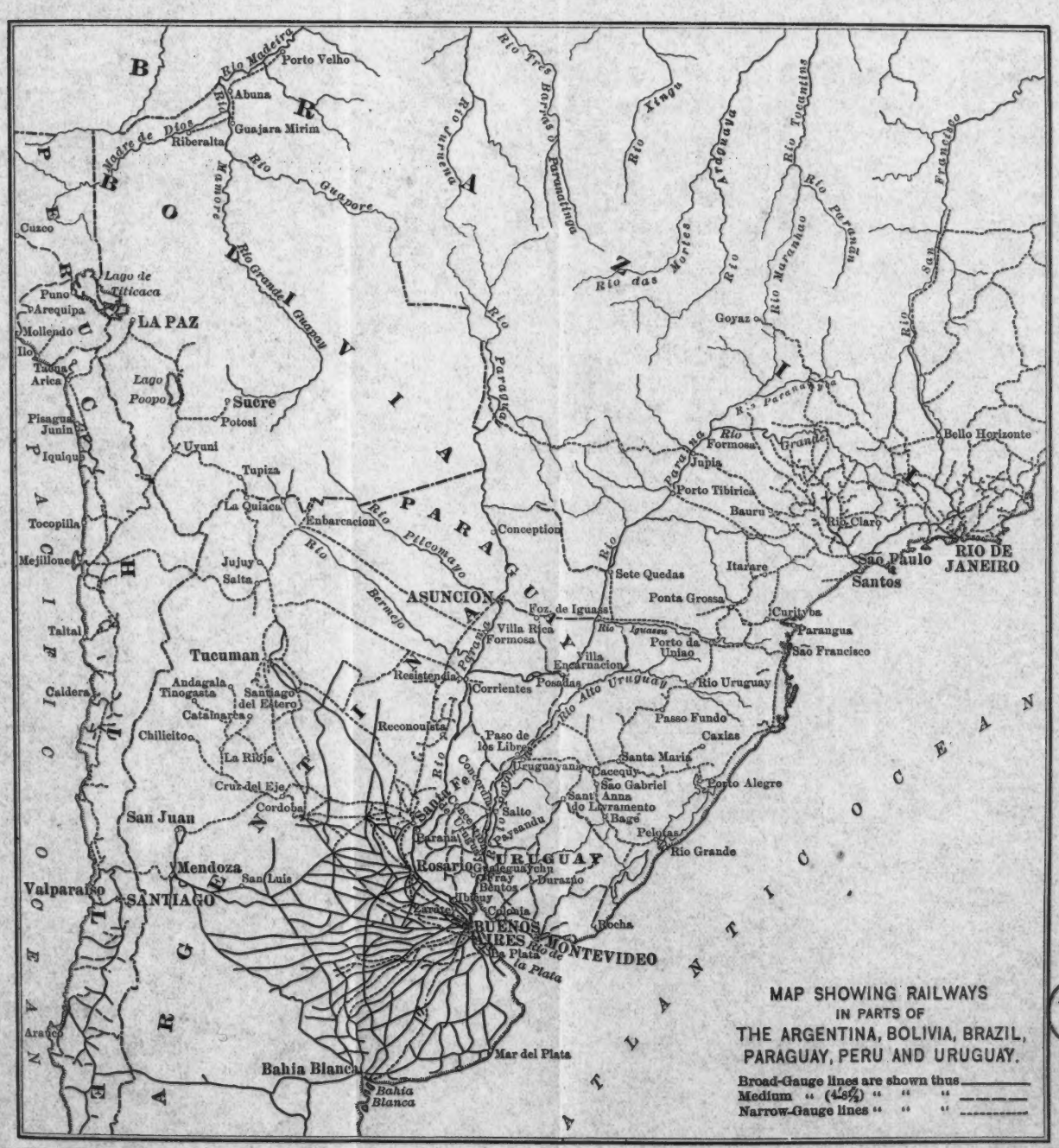
Appendix A contains a table showing the length of the lines of the various gauges throughout the world, the total length of each gauge being, approximately:

Broad	145 300 kilometers	=	90 086 miles.
Medium	727 799 "	=	451 235 "
Narrow	183 416 "	=	113 719 "

The accompanying map, Plate I, shows the railway lines of Southern Brazil, Uruguay, Paraguay, the Argentine, and parts of Chile, Peru, and Bolivia, and indicates the extent of each of the three gauges and their location. Speaking generally, it will be noted that the Brazilian railways are nearly all narrow-gauge, with the exception of parts of the important lines of the São Paulo and Paulista Railways, running into the interior from Santos, the line of the Central Railway of Brazil connecting São Paulo and Rio, and a line from this latter point into the interior, all of these being 5 ft. 3 in. The medium-gauge (4 ft. 8½ in.) lines occupy the strip of country between Brazil and the Parana River, including the whole of Uruguay and Paraguay, and the Argentine Provinces of Entre Rios and Corrientes. In the Argentine, the most intensively developed section—that lying between the City of Bahia Blanca on the south, and the Cities of Rosario, Cordoba, and Mendoza on the north—is occupied principally by the broad-gauge, the narrow-gauge railways occupying the section to the north of this, though in the zone between Buenos Aires, Mendoza, Cordoba, and Santa Fe, the territory is divided between the two, the broad-gauge having somewhat the advantage. Chile and Bolivia seem to be committed to the meter-gauge for all new lines, the former, however, has seven distinct gauges, and in Bolivia the Antofogasta Railway is 2 ft. 6 in. Peru, although fully as mountainous as any part of Chile or Bolivia, has practically adopted the 4 ft. 8½ in. gauge, its lines being among the highest in the world, with several hundred miles at elevations of more than 10 000 ft. above sea level, the famous Oroya Railway reaching 15 583 ft., the highest main line in the world. North of Peru the lines are nearly all narrow-gauge until Mexico is reached, the single exception being the Panama Railway, which is 5 ft.

A more detailed description of the railways of the Argentine, and

PLATE I.
PAPERS, AM. SOC. C. E.
MARCH, 1914.
LAVIS ON
THE GAUGE OF RAILWAYS.



MAP SHOWING RAILWAYS
IN PARTS OF
THE ARGENTINA, BOLIVIA, BRAZIL,
PARAGUAY, PERU AND URUGUAY.

Broad-Gauge lines are shown thus _____
Medium " (4.8½) " " " _ _ _ _
Narrow-Gauge lines " " " ----------

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of the climatic, topographical, general physical conditions, and business of the southern part of South America is given in Appendix B.

Effect of Gauge on Location.—It is quite generally supposed that the narrow-gauge is better adapted to mountain location than the wide-gauge, because it permits the use of sharper curves and steeper gradients. Within the limits of practical railroad operation, this supposition is not warranted by the facts, when comparing meter and 4 ft. 8½ in., and there is very little difference with regard to 5 ft. 6 in.

Curvature.—Within the limits of curvature allowed on the meter-gauge roads of the Argentine, that is, with a minimum radius of 490 ft. (150 m.), it is quite possible to operate 5 ft. 6 in. gauge rolling stock which, as a matter of fact, is handled every day in long trains with side-buffers in the yards at the terminals over No. 8 turnouts which have a radius of about 485 ft. (148 m.), and over main-line, No. 10 turnouts with a radius of 780 ft. (241 m.). There are said to be branches on some of the broad-gauge roads with curves of 490 ft. (150 m.) radius; and the line to the power-station at Lules, near Tucuman, over which broad-gauge trains are operated, has curves of less radius than this. The Southern has just located a line with curves of 656 ft. (200 m.) radius across the mountains beyond Neuquen.

The idea still exists in the minds of many people that trains can actually pass sharper curves on a narrow-gauge than on a broad-gauge. "Reduced to absurdity," this, of course, is so, but within the limits of practical railroad operation, on such lines as those under consideration, it is not, and there can be no question that trains of 5 ft. 6 in. gauge with center couplings can pass around any curves which it is practical to operate on meter-gauge lines built to do business. Sharp curvature limits the speed of trains, and in this respect it imposes quite as much, probably greater, limitation on narrow-gauge trains by reason of their lesser stability than it does on broad-gauge trains. The fallacy that sharper curves are not possible on broad-gauge lines probably arose in the early days, before the swiveling truck was introduced. The possible adaptability of the broader or medium gauge to the conformation of the ground has also quite generally been confused by the assumption that, because the gauge was broader the whole road and rolling stock must be of heavy construction; that is, with many people the term "light" railways has

been considered synonymous with narrow-gauge, whereas there is no reason that light rails and light rolling stock cannot be used as well on broad-gauge as on narrow. One has only to consider the rolling stock in use on British Railways 25 years ago, or even to a large extent in England to-day, with the little 5-, 8-. and 10-ton wagons, to realize this. The use of sharp curvature is by no means advocated as desirable, as the objections to its use are fully realized, but the objections to sharp curvature are no less on meter-gauge than on the medium or broad.

Properly designed rolling stock is, of course, a necessity in any case. Certain 40-ton shunting engines of the Cordoba Central Railway (meter-gauge) have difficulty in passing curves of 328 ft. (100 m.) radius in the yards, by reason of their stiff frames and long wheel base. The 850 000-lb. Mallets on the Atchison, Topeka and Santa Fe Railroad in the United States (4 ft. 8½ in.) are operated on curves of only slightly greater radius without trouble, and haul trains of standard Pullman equipment, the coaches being from 65 to 75 ft. long.

A locomotive of the Mikado (2-8-2) type, having a total weight of 285 000 lb. (220 000 lb. on drivers), built by the Baldwin Locomotive Works, has just been put in service on the railway of the Woodward Iron Company (4 ft. 8½ in.), on a line with 3% grades and 16° (110 m. radius) curves.

The New York City Elevated Railways (4 ft. 8½ in.) were built about 35 years ago, and are operated on an open steel viaduct over the streets of New York. Nearly a million passengers and about 1 000 trains a day pass over them. They have several curves of 90 to 125 ft. (30 to 40 m.) radius, and have an enviable record for safety, only one accident to a train in which the life of a passenger was lost having occurred in 35 years. These lines were formerly operated by steam, but now by electricity with multiple-unit trains.

On the lines of the New York City Subway (4 ft. 8½ in.), built about 5 years ago, an average of more than 1 000 000 passengers are carried daily. On these lines the radius of many curves is 147 ft., over which cars 51 ft. long and 9 ft. 0½ in. wide, are operated in 10-car trains by the multiple-unit system.

Table 2 contains some data for standard-gauge lines operating (many with a very heavy traffic) over sharp curves and heavy gradients,

some of them, as will be seen, rising to quite high elevations and thus indicating the adaptability of the 4 ft. 8½-in. gauge to mountain location.

TABLE 2.—DATA RELATING TO GRADIENTS, CURVATURE, ETC., ON CERTAIN RAILROADS.

	Grade.	Radius of Curves.		Elevation of summit.	Remarks.
		In feet.	In meters.		
Canadian Pacific	4.49%	498	152	5 299 ft. (1740 m.)	.....
" New Line	2.20%	574	175	.....	Spiral tunnels.
Atchison, Topeka and Santa Fé	3.30%	357	109	7 510 ft. (2 464 m.)	425-ton Mallet.
Denver and Rio Grande	3.80%	498	150	10 433 ft. (3 423 m.)	.....
Colorado Southern	3.50%	193	59	.....	Georgetown loop.
Mersey Tunnel	3.00%	.....	....	.....	.....
Mexican Railway	4.75%	.....	....	.....	.....
Nitrate Railway of Chile	4.00%	298	91	.....	.....
Central Railway of Peru	.....	.....	....	15 865 ft. (5 205 m.)	.....

On the Atchison, Topeka and Santa Fé Railroad—a transcontinental line of very heavy traffic—articulated locomotives of the Mallet type, weighing 425 tons (engine and tender), are in use.

Canadian Pacific: The old 4.49% grade has been operated by adhesion since 1885 up to within a year or two ago, when the spiral tunnels on curves of 574 ft. (175 m.) radius were built and the grade reduced to 2.2 per cent.

On the Nitrate Railway of Chile, 4 ft. 8½-in. gauge, a duplex articulated locomotive hauls 200 tons at 8 miles per hour on 3 to 4% grades (average 2.8%) with curves of 300 ft. radius.

In connection with the location of the line of the Wolgan Valley Railway in Australia, the whole question of gauge and the adaptability of standard (4 ft. 8½ in.) gauge to difficult mountain location was carefully studied by the Chief Engineer, Mr. H. Deane, who reported as follows:

"To bring the line within the region of practicable cost, a ruling grade of 1 in 25 was adopted, with curves of 5 chains [330 ft. = 100 m.] radius. There was no possibility of compensating for curvature, and the 1 in 25 grades occur, therefore, on 5-chain curves, so that the actual ruling grade may be said to be 1 in 22.5 not 1 in 25. A study of plan and section, as well as an inspection on the ground, will show how rigid were the conditions of the problem.

"Bound up with the whole question was that of gauge. Steep grades on a narrow gauge limit the load too much. It was anticipated that when the Company was in full swing, over 1 000 tons of goods would have to be conveyed over the line daily. It was clear, therefore, that the standard gauge must be adopted, especially as the Railway Commissioners had offered to lend their rolling stock if that gauge were not departed from. But how about curvature? it will be said. Was it not excessive? No. Not for the wagons, which were

daily hauled safely over the Camden Railway with its 5-chain curves. But what about locomotives? On the Western line, curves of 8 chains [528 ft. = 173 m.] radius were originally constructed, and had all been cut out because the wear of rails and flanges had been excessive.

"This question had to be solved by looking to the practice of other countries. In New South Wales the locomotives were too stiff. Some other type must be adopted.

"During an extensive journey around the world in 1894, I found numerous curves of 16 degrees, equal to 5½ [363 ft. = 111 m.] chains radius, one curve of 18°, equal to 4.8 chains radius and one of 22°, equal to 4 chains radius, on the Southern Pacific Railway system in the Western United States, and these were traversed by 8-wheeled coupled American locomotives of the Consolidation type. This was rendered possible by providing two of the pairs of wheels with broad, plain treads in place of flanging them. The curves mentioned have now been cut out, but they were worked for many years.

"In 1904 I travelled in a train on the main line of the Canadian Pacific Railway where one curve of 3½ chains [231 ft. = 70 m.] radius exists. All the Company's locomotives traverse this curve.

"On some of the mining branches of the Canadian Pacific Railway, where curves of 5 chains and grades of 4½%, equal to 1 in 22.5, exist, Shay locomotives are used.

"On the Tamaulipas Railway, a scenic line in California, there are curves of 70 and 80 ft. [21 to 25 m.] radius, the traffic being hauled by locomotives of the Shay type.

"On the Kandy Railway in Ceylon there are curves of 5 chains radius, the gauge being 5 ft. 6 in. These are negotiated by locomotives built by Kitson and Company, of Leeds. They are 6-wheeled, coupled, with bogie in front. The middle wheels have thin flanges, considerable play in the axle boxes is allowed, and the connecting rod and side rod pins are barrel shaped, so as to permit of the rods working out of the straight line."

As a further concrete example of the possibilities of operating on standard gauge over mountain lines with heavy gradients and sharp curvature, the following may be cited,* the line being part of one of the Transcontinental routes in the United States:

The main line of the Southern Pacific Railroad in California, between Bakersville and Mojave, a distance of about 68 miles (109 km.), is single track, with maximum grades of 2.2%, uncompensated against traffic both ways, and 10° 20′ (555 ft. = 169 m. radius) curves. There is one continuous curve with a total central angle of 566°, nearly all of which has a radius of less than 600 ft. (180 m.), 18 tunnels

* *Railway Age Gazette*, July 25th. 1913.

with a total length of 8 115 ft. (2 473 m.) and about 4 000 ft. (1 219 m.) of bridging. The summit is 4 025 ft. (1 227 m.) above sea level.

The engines used on this line are as follows:

Passenger service, Mogul (2-6-0) type:

Total weight of engine	166 320 lb.
" " on drivers	144 120 "
Tractive effort	33 320 "

Helper engines, Consolidated (2-8-0) type:

Total weight of engine	208 000 lb.
" " on drivers	187 000 "
Tractive effort	43 305 "

Freight engines, Mallet (2-8-8-2) type:

Total weight of engine	435 800 lb.
" " on drivers	401 000 "
Tractive effort	94 880 "

On a single day in January, 1913, 36 trains passed over this section, of which 16 were passenger trains handling 110 passenger cars, 6 305 tons, and 20 freight trains handling 886 cars, 30 725 tons, a total of 36 trains, 996 cars, 37 030 tons. So that, although sharp curvature is not by any means advocated as desirable, it can be shown that where the exigencies of the topography demand its use, standard-gauge equipment can be operated on it quite as well as on meter-gauge, and the possibilities of reducing operating costs by the use of larger engines far outweigh any small economies in the first cost of narrow-gauge by reason of slightly decreased width of roadbed and shorter ties.

Gradients.—As regards gradients, of course, the gauge makes no difference, the same power being required in each case to move the same weight and kind of train. Wider gauge, however, permits the design of larger locomotives, much beyond the limit of capacity of the narrow-gauge, and the wider firebox and freedom from the cramped space of the narrow-gauge is a decided advantage. It is a fact that standard-gauge locomotives are worked on short stretches, such as approaches to coaling stations, etc., by adhesion, on grades as steep as 10%, and Shay geared locomotives regularly on grades of 4 and 5%; and, as noted previously, there are several lines of heavy traffic

worked by adhesion locomotives on grades between 3 and 4%, and until only recently the Canadian Pacific had grades of 4½% on its main transcontinental line worked by ordinary locomotives. The writer knows of no steeper grade than this regularly worked by adhesion on narrow-gauge.

As in the case of curves, steep gradients are not advocated as by any means desirable, but the ability to operate trains on them is not affected by gauge, except in the one very important item against the narrow-gauge of the very limited capacity of the locomotives as compared with the wider gauge. The heaviest locomotive of the Mallet type, built for narrow-gauge, weighs 176 tons, as compared with 425 tons for the heaviest standard-gauge, or in the proportion of 1 to 2.4.

Speed.—At present there is little demand in the Argentine for speeds in excess of those which can comfortably be maintained on narrow-gauge lines in first-class condition, but the growth of the country is so great that it is rapidly reaching the point where fast speeds will have to be maintained over long distances in order to get passengers and mail to their destinations within a reasonable time. There is no reason to think that the Argentine will long remain content with inferior service, but will expect and will get as good service as is obtained in North America, with which region its transportation system will be most closely comparable.

A tabulation of the regular schedule of the principal long-distance and fast trains in the Argentine at the present time shows the maximum and average speeds to be as follows:

	Kilometers per hour: Maximum.	Average.	Miles per hour: Maximum.	Average.
Broad gauge.....	61.8	46.3	38.4	28.8
Medium "	30.0	28.4	18.6	17.6
Narrow "	39.6	29.0	24.6	18.0

It is well known that in Europe speeds of 50 to 56 miles per hour are maintained regularly for distances of 200 miles and more by a large number of trains daily. There are many fast trains in the United States, at least 25 or more daily, which do as well or nearly so, though they are much heavier.

On the transcontinental and other long-distance runs in the United States, varying from 1 000 to 2 500 miles, average speeds of 35 to 40

miles per hour are maintained, and daily trains between New York and Chicago on two roads average more than 50 miles per hour for the 1 000-mile run.

Most of the lines in the Argentine, and practically all those cited, have very light gradients and good alignment, whereas, in the United States, most of the lines, especially the transcontinental lines where they cross the Rocky Mountains, have sections of long heavy gradients with considerable curvature; so that, both actually and comparatively, the Argentine is far behind the United States and Europe in the matter of speed, and the narrow-gauge lines are far behind the broad-gauge. The medium-gauge lines of the Entre Rios and the North East Argentine Railways can hardly be considered for comparison at present, as they are not in good physical condition, but they already have in hand plans looking toward an improved service between Buenos Aires and the North which should enable them to run their trains through to Posadas or Corrientes in about 24 hours, or at an average speed of 30 miles per hour, or faster if necessary.

In a discussion before the Institution of Civil Engineers of Great Britain in regard to the gauge of Indian Railways, and in reply to a question as to whether high speed was necessary in that country, the statement was made that, "India, like every other country, had progressive ideas of speed, comfort, and safety, and these would have to be met." There can be no question, also, that this applies equally to the Argentine and Brazil.

For passenger service the narrow-gauge is hopelessly handicapped, and can never meet the requirements of modern railway service either in speed, comfort, or safety. An ordinary through passenger train in the Argentine will consist of about twelve coaches, part of which will be sleepers, diners, etc. Such a train in the United States would weigh between 700 and 800 tons loaded, but, owing to the generally lighter construction of the Argentine rolling stock, the total weight on the broad- or medium-gauge lines there would be about 500 tons, and on the narrow-gauge, with the same type of rolling stock, it would weigh about 400 tons, with less than three-quarters the capacity.

It is possible, on narrow-gauge lines, in first-class condition, with the best design of engine, to reach a maximum speed of 50 miles per hour on a level grade with a comparatively light train (250 to 300 tons).

On grades even as light as 0.5 to 0.6%, which are the ruling ones on most of the lines of heavy traffic of the Argentine, and are the grades on the Central Cordoba Railway line between Buenos Aires and Rosario, the best average speed would hardly be more than 40 miles per hour, which would mean that the Central Cordoba could not, at the best, make the journey in less than 4½ hours, as compared with the present actual time of the Central Argentine Railway of 5 hours, and the possibility of the latter doing it easily in 3½ hours, if necessary, that is, at an average speed of about 50 miles per hour for the run.

With a train of 400 tons, the sustained speed between stations on the narrow-gauge, with very good track, will not exceed 40 miles per hour, and the average speed of express trains, including stops not oftener than, say, once every 62 miles (100 km.) will not be more than 35 miles. This speed might be adequate, but it can only be attained on almost perfect track, and even then at a considerable sacrifice of the margin between safety and recklessness. The effect of every slight irregularity in the track is felt very much more on the narrow-gauge than on medium or broad, and in this respect the broad has an advantage over the medium in a country like the Argentine, where stone or gravel ballast is so expensive.

Stability.—The lesser stability of the narrow-gauge will be immediately realized when one stops to consider that the height of passenger coaches must be practically the same, no matter what the gauge is. Considering the gauge reduced to a minimum with the height remaining the same, it can be seen that a difference in level of the two rails of, say, ¼-in.—less than that of many low joints in earth ballast—would be sufficient to overturn a sufficiently narrow and high vehicle, whereas it would have very little effect on a wide one, even at high speed. Then again, as soon as unevenness develops in the track, the shock caused by passing trains is greater on the narrow-gauge by reason of the high center of gravity, more height in proportion to width, and becomes worse much more quickly, thus increasing the cost of maintenance and making the hard-riding track so often noticed on narrow-gauge roads. Fig. 1 will, perhaps, help to emphasize this point, which is quite important and often lost sight of by those who compare the efficiency of rolling stock on one gauge or the other.

The growing realization of the fact that the speed and weight of modern trains are producing stresses, even in the very best of modern track, far greater than were formerly suspected, is shown by the action of the Canadian Society of Civil Engineers, which, recognizing the probable existence, due to increased loads, of stresses not satisfactorily distributed by the old short ties, has advocated a longer tie.

The Railroad Commissions of the States of Pennsylvania and New York have ordered a reduction in the speed of the fast 18-hour Chicago trains, due to the difficulty of keeping the track in sufficiently good condition to permit these high speeds. The following is from a recent editorial in a technical journal:*

"It seems to us worth while at this time to enter strong protest against any further increase in the loads which are imposed on the steel rails, track and bridges of American Railways."

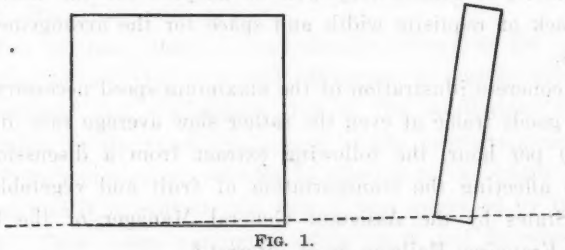

FIG. 1.

In the same journal† there is the following:

"The problem of weight is particularly difficult in regard to passenger locomotives [United States 4 ft. 8½ in.-gauge] * * *. Already we have axle loads of 60 000 to 68 000 lb., and the impact effect or 'hammer-blow' of such loads at high speeds is very severe upon joints, frogs, low spots, and the track in general."

The foregoing comments receive additional force when applied to narrow-gauge roads, where the much greater height in proportion to the width of the vehicles imposes additional shocks and much greater impact stresses than on the wider gauge.

It is well known, of course, that in the transmission of the power from the reciprocating parts of the locomotive engine to the wheels there is an unbalancing of forces which is only partly overcome by the counterbalancing of the wheels. Take, for instance, a Pacific type

* *Engineering News*, December 26th, 1912.
† July 11th, 1912.

engine with 60 000 lb. per axle, running at 60 miles per hour, the rail pressures under each wheel will vary from about 20 000 to 40 000 lb. at each revolution, according to the position of the counterbalance in relation to the rail.

An attempt to overcome this has been made in the class of engines known as balanced compounds, which have been used to a somewhat greater extent in Europe than in the United States. In this type of counterbalanced locomotive the four cylinders are arranged in a row, two inside and two outside the frame, and by the alternating action of the units of each pair they tend to overcome much of the unbalancing of forces noted above. It is hoped that the use of this type will tend to reduce the excessive track stresses with which we now have to contend, but, on account of the necessary arrangement of the cylinders, it cannot very well be adopted for the narrow-gauge, due to lack of requisite width and space for the arrangement of the cylinders.

As a concrete illustration of the maximum speed necessary in order to move goods traffic at even the rather slow average rate of 15 miles (24 km.) per hour, the following extract from a discussion of the problems affecting the transportation of fruit and vegetables in the United States by the Assistant General Manager of the St. Louis and San Francisco Railway, is of interest.*

Citing the run from New Orleans to Denver, 1 515 miles (2 438 km.), he states that at an average speed of 15 miles per hour, which seems quite slow for fruits and vegetables, the total time on this trip would be 101 hours. To get the actual running time there would have to be deducted:

Time lost at 10 district terminals, 1½ hours each	= 15 hours.
Time lost on each of 10 divisions of a single-track road for passing other trains, taking water and coal, say, 2 hours each	= 20 "
Total	35 hours.

This leaves 66 hours actual running time, or an average speed of about 24 miles (39 km.) per hour. In order to make this average speed, and considering the time lost for slow speeds on heavy grades,

* *Railway Age Gazette*, February 7th, 1913.

slowing down for orders (this particularly in the Argentine with the "Via Libre" system), signals, crossings, junctions, etc., makes it necessary to attain a speed of 40 miles (65 km.) per hour at least, on a good proportion of the distance.

To do that with the average freight train necessitates quite good track—it certainly would be verging on the dangerous to attempt it on even fairly good track on the narrow-gauge.

Rolling Stock.

One of the claims in favor of the narrow-gauge most frequently heard is that of the better utilization of the rolling stock; that the width of the cars is greater in proportion to the width of the gauge than it is on the wider gauges; that there is less dead load to paying load, and often that it is cheaper.

The first of these claims is undoubtedly correct, as far as it goes, but whether or not this is a real advantage is questionable, for, as already pointed out, the impact due to the oscillations caused by imperfections in the track (by reason of the proportionately greater height of the meter-gauge vehicles in relation to the width of gauge) is greater, and it is evident that this is also increased by the greater overhang. This causes a rougher-riding track and increased cost of maintenance, the narrow-gauge being forced to the limit in the matter of overhang, in order to provide reasonable capacity. That the other claims cannot be substantiated is believed to be shown by the following discussion.

Passenger Cars.—When it is considered that for passenger trains the capacity of the narrow-gauge is only three passengers in the same length of coach as will provide accommodation for four on either the medium or broad, it is seen at once that the slight extra cost of construction of the latter is more than justified. Comparing the medium and broad, the medium accommodates four passengers quite comfortably, two on each side of the aisle, the broad-gauge does the same, with some slight excess of room, and it is useless for passenger business to consider increasing the width of the broad-gauge, because, to be effective, this would involve three passengers on one side and two on the other, which, besides making unequal loading, would not be comportable and, therefore, is out of the question. In the dining

cars the medium gauge in the Argentine only allows for three passengers in the width of the car, though four can be accommodated by the broad, and this is the only place where the latter takes any advantage of its increased width. The latest type of all-steel diners in the United States, however, accommodates four passengers, two on each side of the aisle. In sleeping cars, whether of the Pullman type, or with compartments with transverse berths, as in general use in the Argentine, the medium-gauge is quite equal to the broad, and has generally less dead weight per passenger. When divided into compartments, the medium is wide enough to provide comfortable berths crossways of the car, and this is all the broad-gauge can do. If the Pullman type of car is used, the broad-gauge only gives more space in the aisle, which is of no advantage, so that the medium is at a slight advantage over the broad in the sleeping cars. On the narrow-gauge, compartments with berths across the cars are really impractical, although some have been built this way and the space quite ingeniously utilized, yet the berths are not long enough to be comfortable for the ordinary man. The narrow-gauge, therefore, is practically forced to the Pullman type of berths, placed lengthwise of the car, but utilizing only one side of it (as compartments have to be provided) so that they only have about one-half the capacity per unit of length of train that the medium-gauge has, and the dead load per passenger in narrow-gauge sleepers is about 2 tons, as compared with 1½ tons on medium or broad.

In order to determine, as far as possible, the relative capacity, cost, etc., of rolling stock on the different gauges in the Argentine, such information as could be obtained was collected, and is presented herewith for what it is worth. It is not as complete as might be wished, and, of course, comparison of rolling stock in any way is only valuable if the type of construction is the same. Practically, it is obvious, however, that given the same type of construction, the spacing of the wheels can make little if any difference, and these actual figures are given simply as confirmation to show that there is little difference in actual practice. The details of the passenger equipment are given in Table 3.

It will be noted that, in regard to cost, although the difference between the cost per ton as between narrow and medium is about 19%.

the difference in cost per passenger accommodated is 34%, both in favor of the medium-gauge.

Freight Cars.—The advocates of the narrow-gauge, though generally admitting that for passenger business the broader gauges have some advantage, almost invariably claim an equality in transportation of freight, within the limits of their capacity, and that their rolling stock is cheaper and has less dead weight in proportion to carrying capacity.

TABLE 3.—DETAILS OF PASSENGER EQUIPMENT.

	NARROW.			MEDIUM.			BROAD.		
	Weight, loaded, in tons.	Seats.	Cost, in gold.	Weight, loaded, in tons.	Seats.	Cost, in gold.	Weight, loaded, in tons.	Seats.	Cost, in gold.
First-Class.....	28	47	$12 250	38.3	68	$12 475	33.8	56	$11 800
Second-Class.....	27	67	9 900	40.5	102	9 700	32.2	64	8 590
Diners	61	48	29 000	37.7	40	17 000	33.6	32	13 050
Sleepers.....	27	12	11 760	37.0	20	15 520	32.2	23	12 770

COMPARISON OF UNITS.

Cost per ton.....	$440	$356	$350
Cost per passenger.....	361	238	264
Weight per passenger, in tons..	0.82	0.67	0.75

It is generally claimed that in the Argentine there is not much demand at present for either very long trains or cars of large capacity. Leaving this phase of the question for the present, it will be of interest to compare the freight cars of the three different gauges on somewhat the same basis as the passenger cars.

In regard to certain types, some additional data have been furnished to the writer by the Middletown Car Company, which has supplied a large number of narrow-gauge cars to the Government lines, and also to a standard-gauge line in Uruguay and to the broad-gauge port railway at Buenos Aires, these cars being of the same general types of construction. It is to be noted that the comparison of freight cars is somewhat difficult by reason of the greater variation in types of construction, the proportion of tare to total capacity

varying from 40% for well-designed cars to as much as 60% in some of the older types of less capacity. In the United States, also, the cars of large capacity show up to much better advantage than those of smaller capacity, as the underframes and draft rigging have to be the same for all, in order to allow all types of cars to be used in heavy trains. As far as possible, however, from the data of a large number of cars examined, the following are selected as representing good average practice in the Argentine for well-designed cars of fairly large capacity.

The question of what the result would be if the broad-gauge cars were changed so as to get the same proportionate overhang as medium-gauge, is discussed later, and the following comparisons are made on the basis of existing conditions, which it is unlikely will be changed.

It is to be noted that the greatest variation is to be found in the cost of the cars of the Rosario Puerto Belgrano Railway, which is considerably above the average. It is believed that, if sufficient information could be obtained as to the cost of freight cars on the other principal broad-gauge roads of the country, it would show little variation from the costs given of narrow and medium-gauge. The tons are metric, of 1 000 kg. (2 205 lb.).

TABLE 4.—Data Relative to Covered Wagons (Box Cars).

	Capacity, in tons.	Area of cross-section, in square feet.	Cubic feet.	Tare, in kilogrammes.	Cost.
Narrow.					
Middletown Car Company	30	51	1 695	12 200	.....
Central Cordoba	25	45	1 340	10 500	$1 386
Central Northern	20	.....	.....	10 000	1 100
Medium.					
Middletown Car Company	30	70.7	2 545	15 380	.....
Entre Rios	30	73.4	2 350	12 500	$1 470
Broad.					
Middletown Car Company	30	71.1	1 860	12 675	$1 400
Rosario Puerto Belgrano	40	75.1	2 885	.....	2 075
Southern	40	.....	2 220	15 120	1 875
Central Argentine	40	.....	2 185	16 350	1 875

It will be seen at once that, as far as regards weight of car to capacity in tons, there is little difference, with the exception of the Middletown Car Company's medium-gauge car, which has very large cubic capacity, the 30-ton cars averaging about 40 to 42 per cent. These small differences are less than are to be found between cars on any one line, of different types of construction.

It will be noted, however, that the narrow-gauge cars have generally only about two-thirds the cubic capacity of the wider gauges. The Middletown Car Company's broad-gauge car is quite a short one, being only 26 ft. long (the usual length being 36 ft., more or less), and, as a large proportion of the goods hauled in box cars is bulky rather than heavy, the extra cubic capacity is a most decided advantage. In the Argentine, as nearly as can be calculated from the statistics published by the Government, more than one-half the freight generally handled in box cars is of the former class, that is, classified by bulk rather than weight.

TABLE 5.—Data Relating to Stock or Cattle Cars.

	Capacity, in tons.	No. of animals.	Width, inside.	Floor area, in square feet.	Tare, in kilogrammes.	Cost.
Narrow.						
Middletown Car Company	30	.....	7 ft. 11 in.	231	11 984	.....
Central Northern	20	16	.....	190	10 768	$1 240
Central Cordoba	22	20	8 ft. 0 in.	232	12 000	1 365
Medium.						
Middletown Car Company	26	.....	8 ft. 5 in.	295	15 380	.....
Entre Rios	22	20	7 ft. 10 in.	256	12 000	$1 475
Broad.						
Rosario Puerto Belgrano	40	24	7 ft. 3 in.	281	.....	.....
Pacific	28	.....	9 ft. 6 in.	323	16 000	.....
Pacific	28	.....	9 ft. 6 in.	314	17 525	.....
Average of several	40	.....	8 ft. 6 in.	295	16 250	.....

Stock cars are comparable on the basis of the relation between the floor area and the tare weight. The data in Table 5 are all the specific examples the writer has been able to get, and, in spite of being taken

from various sources, they show considerable agreement, the relation between floor area and tare weight for the three types being as follows:

Narrow	51.9	kg.	per	sq.	ft.	(114 lb.)
Medium	49.7	"	"	"	"	(109 ")
Broad	46.9	"	"	"	"	(103 ")

It may be noted that the Entre Rios car is very small for a stock car for 4 ft. 8½-in. gauge, and is not at all representative of the best practice, which would be at least equal to the others. The prices per ton of car and per square foot of floor area work out as follows:

Narrow: Central Northern	$115.15	per ton	$6.75	per sq. ft.
" Central Cordoba	113.75	" "	6.40	" " "
Medium: Entre Rios	118.75	" "	5.75	" " "
Broad: Rosario Puerto Belgrano	.....	.....	.....	.....

TABLE 6.—DATA RELATING TO GONDOLAS.

	Capacity, in tons.	Floor area, in square feet.	Cubic feet.	Tare, in kilogrammes.	Cost.	
NARROW.						
Middletown Car Company	30	250	804	10 900	.....	.....
Central Cordoba	25	244	.....	11 000	$1 000	.....
Middletown Car Company	30	250	.....	12 210	1 345	All steel.
Middletown Car Company	20	.....	.....	9 070	840	.....
MEDIUM.						
Middletown Car Company	30	292	.....	12 845	.....	.....
Entre Rios	30	284	.....	11 250	$1 250	.....
Rosario Puerto Belgrano	37	308	700	14 400	.....	.....
BROAD.						
Middletown Car Company	45	.....	.....	13 800	.....	.....
Central Argentine	42	336	788	15 240	.....	.....
Western	45	340	1 630	15 200	.....	.....

For gondolas the comparison is best made between floor areas and tare weights. The cubic capacity is affected by the height of the sides, and as these may vary quite a little, this is not made, as all the details

are not available, also the loads in these cars are often heaped up above the sides.

Narrow ..... 45.8 kg. per sq. ft. of floor area (101 lb.)
Medium ..... 43.5 " " " " " " " (96 ")
Broad ..... 47.5 " " " " " " " (105 ")

TABLE 7.—DATA RELATING TO FLAT CARS.

	Capacity, in tons.	Floor area, in square feet.	Tare, in kilogrammes.	Cost.	
NARROW.					
Middletown Car Company	30	262	9 300	.....	.....
Central Cordoba	25	256	8 500	$825	.....
Central Northern	20	217	8 145	.....	.....
Chile	25	257	8 161	.....	All steel.
MEDIUM.					
Middletown Car Company	30	309	11 570	.....	.....
Entre Rios	30	258	11 000	$1 150	.....
BROAD.					
Rosario Puerto Belgrano	40	359	.....	$1 915	.....
Average of seven cars	41	328	13 706	.....	.....

Narrow ..... 34.0 kg. per sq. ft. of floor area (75 lb.)
Medium ..... 40.0 " " " " " " " (88 ")
Broad ..... 41.7 " " " " " " " (92 ")

It will be noted that, in comparing these four types of freight cars, so far as these figures show, there is very little difference between those of either gauge, what difference there is, however, being in favor of the wider gauges, especially for the box and cattle cars. The data are not as complete as might be wished, but are all that are available of Argentine rolling stock at this time, and, in view of the fact that the information has been taken just as it came, it is believed that it confirms what must be seen to be theoretically correct, that is, that gauge can make very little difference in cars of small capacity. A narrow-gauge box car with the wheels spread out can be used perfectly well on broad-gauge tracks which only differ from its own in being slightly wider. This extra width of trucks, as between narrow and

medium, only involves about 200 to 250 lb. more iron and steel, which at 6 cents per lb. is from $12 to $15, and this extra width is more than compensated by the increased stability and, consequently, lesser impact on the track.

The claim, therefore, that narrow-gauge cars are either cheaper or lighter in weight than cars of the same capacity and same type of construction for wider gauges, does not appear to be sustained by the facts, such variations as are shown being less than those between many cars of different make and design on the same railroad. It is of little value to include for comparison data relating to ordinary United States rolling stock, as this is all generally of heavy type, all the underframes and draft rigging being much heavier than those in use elsewhere, in order that they may withstand the strains due to the use of the cars in heavy trains, and, in addition, all cars are equipped with automatic power brakes, which are not in general use in the Argentine. The railways of the latter country, however, are beginning to realize the values of the saving due to the use of heavier cars and trains, and perhaps more so on the narrow-gauge lines. They are handicapped, however, by the fact that the underframes and draft rigging of all their old rolling stock, and even much of the new, are too weak to stand the strain. They are just beginning to be forced into the same position as the railroads of the United States were some time ago, that is, to adopt the same heavy type of underframe and draft rigging, no matter what the size of the car is, in order to be able to transmit the tractive forces without danger of breaking the train. When this becomes general practice, there will be a much greater discrepancy in the relations between the gross and paying loads for the lighter cars, and they will show up at a much greater disadvantage than they do even now.

Cost of Locomotives.—It is practically impossible to determine with mathematical accuracy the relative economy of locomotives for one gauge or another, as they vary so much in detail. Even supposing three machines as exactly alike as possible, except for the difference in the width apart of the wheels, which would give a difference in cost of only a trifling extra quantity of steel for the wider spacing, the performance would be so different as to vitiate any comparison. Even supposing three machines designed to be as nearly alike as possible

but adapted to the gauge, there would still be so many differences that comparison would be questioned.

It seems to be a perfectly safe assertion, however, that, within the limits of the narrow-gauge, any reputable locomotive builder will undertake to build locomotives of each gauge having about the same power, capacity, and relative fuel economy for about the same price, irrespective of the gauge. There may be a little less steel in the narrow-gauge, but the labor for this slight difference is little, if any, less, and the greater ease of design for the broad-gauge more than compensates for this. There seems to be a vague idea in the minds of some advocates of the narrow-gauge that, by some means or other, narrow-gauge locomotives are more economical, whereas it is probable that the contrary is the case, as is noted in detail later. The hauling capacity of a locomotive, provided of course it is properly designed for power, is determined by the weight on the driving wheels, and a locomotive with 10 tons weight on its driving wheels will have exactly the same adhesion on a wide-gauge that it has on a narrow-gauge, so that, to pull the same gross load, the engines must have the same weight, if they are built on the same plan. A pound of coal, too, will not generate any more steam on the narrow-gauge than on the wide.

In order to get some idea at least of the comparative cost of locomotives for the three gauges in the Argentine, the writer obtained the figures given in Table 8, which, considering the different types of engines involved, different makers, etc., show a sufficient agreement in cost per ton at least not to disprove the argument that gauge makes little, if any, difference. The weights are per ton of engine and tender, light.

Size of Locomotives.—As at present worked, the 5 ft. 6-in. gauge has about the same capacity as the medium, though much heavier engines and trains are operated in the United States on the medium-gauge than elsewhere on any gauge. For reasons to be given later, it is believed that the capacity of the broad-gauge, measured by the size of engines and length of trains, will not be increased beyond that of the medium, unless by some entire change in transportation methods not even guessed at now.

The comparisons in Table 9, therefore, between the largest engines built to date (January, 1913), of the three types most generally in

use, will give a good idea of the tremendous advantage of the wider gauge in this respect for lines of heavy traffic.

TABLE 8.—DATA RELATIVE TO LOCOMOTIVES.

Description.	Class.	Cost.	Weight, in tons.	Cost, per ton.
METER GAUGE.				
Kitson, 1911	4-6-0	$12 500	50	$250
North British, Passenger, 1907	4-4-2	18 750	75	250
" " Cargo, 1907	4-6-2	19 500	78	250
Tank	0-6-4	11 500	47	245
Buenos Aires, Extension Passenger, 1906	4-6-2	18 000	78	230
" " " Cargo, 1906	4-8-0	17 250	73	235
Building Pacific Superheater	4-6-2	19 700	67	295
MEDIUM GAUGE.				
North British, Goods	2-8-0	$19 000	68	$280
" " Mixed Passenger	4-6-0	16 000	65	245
North American, Goods	2-8-0	17 000	56	300
" " Mixed	4-6-0	11 900	56	210
Shunting Engines	0-6-0	8 800	26	335
BROAD GAUGE.				
Tank	2-6-0	$9 360	39.5	$237
"	0-4-0	15 550	63.2	282
Passenger	4-4-0	21 400	70.1	302
Cargo	2-8-0	30 310	91.5	331

In the comparisons in Table 9 the 3 ft. 6-in. engines are given for the Pacific and Mallet types as they are the largest narrow-gauge engines, the largest meter-gauge engines of these types thus far built, however, are quite a little smaller, the meter-gauge Pacific type being 19 by 26 in., with a total weight of engine and tender of 229 500 lb. and the Mallet 18½ by 29 by 22 in., with a total weight of 323 500 lb.

The freight engines of both types on the medium-gauge have practically double the capacity, and though the superiority of the passenger engine in tractive power is not so great, the grate area is just double, 70 sq. ft. as compared with 35 sq. ft., and the total heating surface, 3 936 sq. ft. as compared with 1 981 sq. ft., so that the capacity for sustained work, so necessary in passenger service, is very much greater in proportion. Size of boilers, heating surface, and grate areas are the limiting factors, of course, in all types of locomotives, and though the limit seems to have been nearly reached in these items in locomotives for narrow-gauge, it does not seem to be in those for medium-gauge. It

TABLE 9.—COMPARISON OF LOCOMOTIVES.

Type of Locomotives.	Gauge.	Weight of engine and tender, in pounds.	Weight per driving axle, in pounds.	Diameter of driving wheels, in inches.	Cylinders, in inches.	Maximum tractive power, in pounds.	Maximum load, 0.6% grade, in tons.	Remarks.
Passenger, Pacific, 4-6-2 type	4ft. 8½ in.	469 190	57 440	74	24 by 32	49 000	2 350	Standard-gauge: Baltimore & Ohio R. R. *Engineering News*, July 11th, 1912, p. 49.
	Meter.	259 800	35 330	62	21 by 28	28 800	1 584	3 ft. 6-in. gauge, South African Railways, Am. Loc. Co. Catalogue, p. 35.
Freight or mixed, Mikado, 2-8-2 type for fast or heavy service	4ft. 8½ in.	539 480	53 300	63	28 by 32	27 460	3 196	Standard gauge: Erie R. R., 1912. *Engineering News*, July 11th, 1912, p. 51.
	Meter.	246 600	27 625	49	20 by 26	28 900	1 564	Meter-gauge: Am. Loc. Co. Catalogue, p. 23.
Freight, articulated (Mallet), for heavy, slow service	4ft. 8½ in.	850 000	55 000	57	28 by 32	111 600	6 394	Standard-gauge: Atchison, Topeka & Santa Fe R. R. *Engineering News*, May 4th, 1911, p. 548.
	Meter.	352 000	32 000	46	18 by 25½ by 26	57 700	3 266	3 ft. 6-in. gauge: South African Railways. Am. Loc. Co. Catalogue, p. 77.

is stated that there is under construction now a medium-gauge locomotive of 160 000 lb. tractive power, nearly 50% more than that of the large Mallet given in Table 9.

Relative Capacity of Wide- and Narrow-Gauge Lines.—The comparisons between the capacities of the meter and broad-gauge lines of India given in Table 10 were made by Sir Robert Upcott,* the figures covering sixteen of the principal railway lines.

TABLE 10.—Carrying Capacities, Compiled from Railway Administration Report, Covering Sixteen Lines in India: Eight of 5 ft. 6-in. Gauge and Eight of Meter-Gauge.

		Ratio. Standard-gauge to meter-gauge.
Vehicle capacity only	Passenger	1.5 to 1
	Goods.	1.6 to 1
Same, taking speed into account	Passenger	1.9 to 1
	Goods.	1.9 to 1
Gross weight of trains	Passenger	1.4 to 1
	Goods.	2.0 to 1
Same, taking speed into account	Passenger	1.9 to 1
	Goods.	2.4 to 1
Vehicle mileage, loaded and empty	Passenger	1.9 to 1
	Goods.	2.8 to 1
Same, taking speed into account	Passenger	2.4 to 1
	Goods.	3.3 to 1
On actual number of passengers carried 1 mile		2.0 to 1
On actual number of tons carried 1 mile		4.7 to 1

Engines on broad-gauge, average 7 838 000 ton-miles per annum.
Engines on narrow-gauge, average 3 962 000 ton-miles per annum.

The following extract gives the opinion of an important official of the inferiority of the narrow-gauge lines in India, where the problem is very much the same as it is in the Argentine, namely, the movement of very large quantities of grain in a short space of time.

"It has been clearly brought out, writes Major-General Kennedy, by the facts of the present season's famine traffic, that occasions such as the present may arise when, in order to feed a large population, grain has to be carried over single lines of railway for long distances, in quantity sufficient to strain to the utmost the resources of the best appointed lines of the standard 5½-ft. gauge and, as it may be assumed that narrow gauge lines would be far less efficient, the unsuitability of the metre gauge for long lines of through communication cannot be put out of sight when thought is taken for the proper and efficient administration of a country liable to be affected with drought and famine.

* "The Railway Gauges of India," *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV, p. 202.

"Although the railway lines concerned in the present case, *i. e.*, the East Indian, the Great Indian Peninsula and the Madras Railways, have failed to do all that was required of them, or that was needful to meet the full necessities of the case, they have, nevertheless, it may be truly said, largely contributed to save Southern India from a very great calamity, but they have not done this with any margin to spare, or without some injury to particular sections of general trade or to individual traders. The standard gauge lines have been barely able to satisfy essential and urgent demands, and under similar circumstances narrow gauge lines would probably have broken down altogether. Indeed, it is known that in the Southern part of the Madras Presidency considerable inconvenience arose at a break of gauge.

"We may, with the memorial of the Bombay Chamber of Commerce before us, take it as proved that narrow gauge lines are unable to meet the sudden demands likely to be made upon them in India, and that they are far more expensive to work than broad gauge lines. At first sight it is difficult to see how the latter statement can be true, but we think we can clear up the point in a moment. When a given quantity of goods, say grain, has to be conveyed over a railway, two systems of conveyance may be adopted, that is to say, we may run large trains at long intervals or small trains more frequently. Now, on the broad gauge lines in India a goods train complete weighs on the average 505 tons, but on the Rajpootana State metre gauge railway the average goods train weighs but 200 tons, so that the broad gauge trains are really 2½ times as heavy as the narrow gauge trains, while their speed is higher."

Economy of Heavy Engines.—The following statement, recently made by J. J. Hill, F. Am. Soc. C. E.,* one of the foremost railroad executives in the United States, shows the value and importance of heavy engines and wagons of large capacity for the development of cheap and efficient transportation:

"Heavier rails, larger engines, cars of greater capacity, increased train movement and the full utilization of equipment have kept business moving. The density of traffic in England, France and Germany should be as much greater than in the United States as the density in the middle exceeds that in the far western states. Yet here are the facts:

"Ton Miles per Mile of Road.	
France	496 939
United Kingdom	529 622
Germany	827 400
United States (1910)	1 071 096

* *Railway Age Gazette*, December 20th. 1912.

"* * * Our railroads move 272 ton miles of freight per dollar of net revenue, where the United Kingdom shows only 58, Germany 172 and France 88.

* * * * * * * * *

"Transportation costs the public from one-third to one-half as much here as in Europe. This cheapness is not purchased at the cost of the workingman. In 1910 the average daily earnings of railway employees in the United States were more than twice as great as in the United Kingdom, and two and three-quarter times as much as on the Prussian-Hesse system in Germany."

There has been some criticism of the exact figures used by Mr. Hill, and of the use of certain units as indices of efficiency, but, making every allowance for this and the fact that such criticisms of details are always possible, in view of the different conditions and manner of computing and compiling statistics, there is not enough difference to affect the main argument, which is, that the use of heavy rolling stock has been the principal cause of the very much lower costs of transportation in the United States than elsewhere.

Fig. 2,* relating to the improvement in operating efficiency of the Chicago, Burlington and Quincy Railroad during the past 10 years, shows with great clearness the possibilities of reduction in operating expenses by the increase of train loads. During this period, as shown by the diagram, the business of the road was nearly doubled (98.3% increase), though there was at the same time an actual decrease of 9.8% in the number of trains required to handle it. Part of this good showing was made by increasing the car loading, that is, the revenue-tons per car-mile, by 45.5% and the remainder was due to increase in the capacity of the locomotives, reduction of grades, and the maintenance of the roadbed and equipment in a high state of efficiency. The average capacity of cars was increased from 23 to 38 tons. The heaviest type of locomotive in use in 1901 developed about 20 000 lb. tractive power; the Mikado (2-8-2) type now in general use has a tractive power of 60 000 lb. The average train load is now 438 revenue-tons per train-mile, as compared with 200 tons in 1901.

Although Canada is somewhat behind the United States in the matter of economical handling of freight, it is far ahead of any other country in this respect. In New South Wales, where they are vig-

* *Railway Age Gazette*, January 18th, 1913.

orously agitating the double-tracking of many of their lines, the traffic density is only 226 906 ton-miles per mile as compared with 731 776 for Canada, the average train loads being 90 tons and 325 tons, respectively, so that actually more trains are being run in New South Wales to handle a smaller volume of traffic than in Canada, the freight-train-miles per mile of line in New South Wales being 2 512 as compared with 2 252 in Canada, in spite of the fact that the density on the latter is 220% more than on the former, and the average freight charge in Canada is 0.75 cent as compared with 1.78 cents in New South Wales.

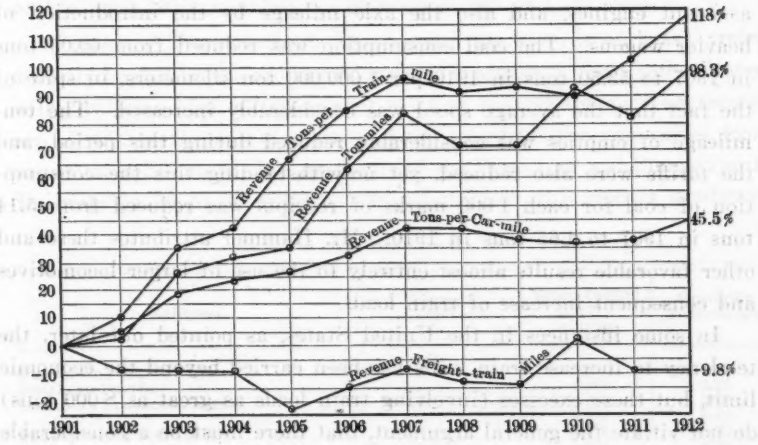

FIG. 2.

It is not only in the United States and Canada, however, that the introduction of heavier locomotives, and consequent increase in train loads, has been found profitable and economical. About 10 years ago, as the result of investigation into the so-called "American" methods of operation, Sir George Gibb, then Manager of the North Eastern Railway of England, introduced changes on that line which have resulted in the increase of the average freight train load from 60 to 95 tons, and on mineral trains from 114 to 184 tons. The result of this* is seen in the fact that for the last half of 1912 a dividend was announced at the rate of 7½% per annum, more than twice the average for British railways.

* *Railway Age Gazette*, March 28th, 1913.

In Germany, also, according to a statement of Mr. Hammer,* there is a growing realization of the necessity for the use of larger locomotives. On the Prussian State Railways (as elsewhere), owing to the continued tendency to increase the dead weight of passenger equipment, both actually and per passenger, in order to provide the better accommodations demanded by the traveling public and also the greater strength required for increasingly high speeds, the average weight per seat on passenger equipment has increased from 0.27 to 0.37 tons, or 37%, in the last 20 years. The introduction of heavier locomotives has reduced the train and locomotive mileage by eliminating many assistant engines, and also the axle mileage by the introduction of heavier wagons. The coal consumption was reduced from 60.05 tons in 1907 to 53.50 tons in 1910 per 1 000 000 ton-kilometers, in spite of the fact that the average speed was considerably increased. The ton-mileage of empties was considerably reduced during this period, and the tariffs were also reduced, yet notwithstanding this the consumption of coal for each 1 000 marks of receipts was reduced from 5.14 tons in 1907 to 4.65 tons in 1910. Mr. Hammer attributes these and other favorable results almost entirely to the use of larger locomotives and consequent increase of train loads.

In some instances in the United States, as pointed out later, the tendency to increase train loads has been carried beyond the economic limit, but these excesses (involving train loads as great as 8 000 tons) do not vitiate the general argument, that there must be a considerable increase in the Argentine to meet the necessities of modern transportation and to carry bulk freight long distances at rates which will enable the hitherto undeveloped sections in the interior of South America to enter the markets of the world, and that the necessary increase is not possible on the narrow-gauge.

Saving in Fuel.—The saving in coal consumption, made possible by the use of some of the more recent types of heavy engines, is also shown by the following quotation from a recent paper† by Mr. O. S. Beyer, Jr. In commenting on fuel, he said:

"Numerous tests and service records have revealed that large superheater Mikado locomotives which have been placed in service recently haul trains of 45 and 50% greater tonnage with the same amount of

* *Bulletin*, International Railway Congress, March, 1913.
† Am. Inst. of Mech. Engrs., Vol. 34, 1912, p. 1301.

coal that was formerly consumed by the Consolidation locomotives they replaced. Even the coal consumption of Mallet engines with grate areas up to 100 sq. ft. has not grown in any way proportionate to the increase in their hauling capacity. Modern engines when running at shortened cut-offs over those portions of the road other than the ruling grades exhibited a still greater economy than when working on the heaviest grades. Some service tests of recently built Mikado engines on the Delaware, Lackawanna and Western Railroad clearly demonstrated these facts. Their economy in fuel consumption as compared with that of the old Consolidation type, both operating over heavy grades at full load, being 20 per cent. The economy effected over easy grades while running at shortened cut-offs was 39.3 per cent. almost twice as much. The average was 29.1 per cent."

Weight of Trains.—Table 11, showing the records of heavy trains actually hauled in the ordinary course of business in the United States, will give some idea of the present state of the art there. The records are all within the past 3 years, and the length of the run varies from about 125 miles on the low gradients of the Virginian and Pennsylvania Railways to 50 to 65 miles on the other lines with steeper gradients.

TABLE 11.

Railroad.	Type of locomotive.	Train behind tender, in tons.	Gradient. Percentage.
Denver and Rio Grande	Mallet	500	4.00
Southern Pacific	"	1 250	2.20
Canadian Transcontinental	"	1 212	2.20
" "	"	4 290	0.40
Pennsylvania Railroad		8 778	0.23
" "	Consolidation	7 644	0.20
Virginian Railroad		6 023	0.20
" "	Mikado	7 562	0.20
Lake Shore and Michigan Southern		7 433	

The last five trains carry coal or iron ore in all-steel gondolas, all of the same type and of 100 000 lb. capacity, having 85 to 100 cars in a train.

Wider Rolling Stock for Broad-Gauge.—This brings us to the consideration of possible benefits to be derived from the broad-gauge, if advantage should be taken of the extra width of track to get the same proportionate width of cars as the medium- or narrow-gauge.

Experience has shown that the width now in use on the meter-gauge, combined with the height of passenger coaches, box cars, and cattle cars, is practically at or even beyond the limit. One has only to travel

on meter-gauge lines, even where the track is well kept up, to realize that as soon as any attempt at speed is made the oscillations are quite violent, and if the track is out of line and surface, speed becomes positively dangerous.

On the standard-gauge, which is well named the normal in many countries, the happy medium appears to have been most nearly attained by proper adjustment of the size of the vehicles to the width of the track. The clearance diagrams, Fig. 3, which are those established for the Argentine, show the ratios in Table 12 between the clearances on the three gauges; and it may be noted that the clearances for medium-gauge are ample for the largest equipment in use in the United States.

TABLE 12.

Gauge.	Permissible width. Argentine.	Width of track.	Ratio.
Broad	11 ft. 1¾ in.	5 ft. 6 in.	2.0 to 1
Medium	10 ft. 10 in.	4 ft. 8½ in.	2.3 to 1
Narrow	10 ft. 6 in.	3 ft. 3⅜ in.	3.2 to 1

TABLE 13.—RATIOS BETWEEN THE ACTUAL WIDTHS OF VEHICLES AND WIDTHS OF TRACK.

Gauge.	Actual width of vehicle.	Width of track.	Ratio.
Broad	10 ft. 6 in.	5 ft. 6 in.	1.91 to 1
Medium	10 ft. 2 in.	4 ft. 8½ in.	2.16 to 1 (Freight cars only 9 ft. 4 in.)
Narrow	9 ft. 1 in.	3 ft. 3⅜ in.	2.77 to 1

The statement is often made—and at first sight apparently with a certain amount of reason—that the broad-gauge has not taken proper advantage of the additional width of gauge. It has already been shown that there would be no advantage in increasing the width of passenger coaches. Up to the present there has been little demand for freight cars of greater capacity than 100 000 lb. (45 000 kg.), but, if there were, it could be met, up to at least 20 to 30% more, without increasing the width of any type. The only possible advantage of the increased width would be that, theoretically, a reduction in cost would be possible, because the nearer the body of the car approches a square in plan the more economical it is to build, and in this the 5 ft. 6-in. gauge has a

superiority over the meter-gauge which more than compensates for the cost of the additional width of track. Between the present width of the broad-gauge rolling stock (10 ft. 6 in.), however, and the width to make it proportionately equal to the medium-gauge, say 12 ft., there

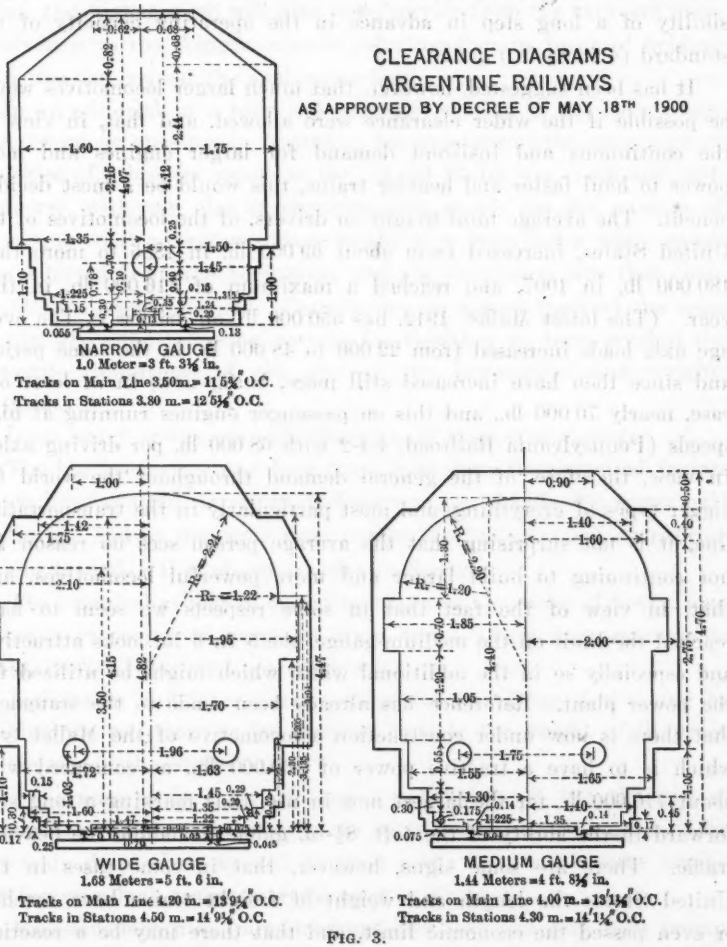

FIG. 3.

would be little saving in the cost of the vehicles, and no advantage in carrying capacity, even though the increased cost of additional clearance required for overhead structures, tunnels, etc., is practically negligible in the Argentine, as there are so few. There has recently been

designed for the Norfolk and Western Railway, and placed in service, high-sided steel gondolas of 100 tons capacity, weighing light about 65 000 lb., and having a simplified form of six-wheel truck. These cars are 10 ft. 4½ in. wide, over all, and their use indicates the possibility of a long step in advance in the operating capacity of the standard (4 ft. 8½ in.) gauge.

It has been suggested, however, that much larger locomotives would be possible if the wider clearance were allowed, and that, in view of the continuous and insistent demand for larger engines and more power to haul faster and heavier trains, this would be a most decided benefit. The average total weight on drivers, of the locomotives of the United States, increased from about 69 000 lb. in 1885 to more than 180 000 lb. in 1907, and reached a maximum of 316 000 lb. in that year. (The latest Mallet, 1912, has 550 000 lb. on drivers.) The average axle loads increased from 22 000 to 48 000 lb. in the same period, and since then have increased still more, being now, in at least one case, nearly 70 000 lb., and this on passenger engines running at high speeds (Pennsylvania Railroad, 4-4-2 with 68 000 lb. per driving axle). In view, therefore, of the general demand throughout the world for bigger types of everything, and most particularly in the transportation line, it is not surprising that the average person sees no reason for not continuing to build larger and more powerful locomotives, and that, in view of the fact that in some respects we seem to have reached the limit on the medium-gauge, the 5 ft. 6 in. looks attractive, and especially so in the additional width which might be utilized for the power plant. Reference has already been made to the statement that there is now under construction a locomotive of the Mallet type which is to have a tractive power of 160 000 lb., as compared with about 110 000 lb. for the largest now in use, thus marking a long step forward in the ability of the 4 ft. 8½-in. gauge to handle the heaviest traffic. There are some signs, however, that in some cases in the United States the length and weight of freight trains have reached or even passed the economic limit, and that there may be a reaction from the extremely heavy loadings, though by no means to reduce them to anything like the low standards of Europe or South America.

It has been pointed out* that these extremely large engines have

* *Engineering Record*, February 20th, 1909.

greatly increased the cost of bridges, and that heavy locomotives, because of the increased stresses in the track, increase the cost of maintenance. High speed, combined with heavy weights, is the cause of far more rail failures than poor steel. Lighter locomotives, shorter trains, and higher speed will give more service from the cars and more satisfaction to the shipper, who is not altogether in favor of holding trains for the last car the engine can haul.

In an editorial in a technical journal* relating to the controversy over the gauge question in Australia, the question of the advantage or otherwise of a broader gauge in the United States was examined; not, of course, with any idea of changing, but purely for the purpose of discussing what advantages there might have been had a wider gauge been adopted from the beginning, and the conclusion was that there would have been no benefit. Considering the matter further† the same journal gives the following reasons for thinking we have reached the economic limitations of the size of the locomotives and weights of trains:

Long heavy trains are only desirable if they save money.

The big locomotive and long train have been the principal factors which have enabled American Railways to pay double the wages paid in Europe and at the same time reduce the ton-mile cost of hauling freight far below that of any country in Europe.

The principal saving in operating expenses by reason of heavy, long trains is in saving in wages of train crews, which item amounts to about 12½% of the total operating cost.

Legislation and labor organizations are nullifying this saving by making the pay more or less proportionate to the weight of the train or the size of the engine.

An argument in favor of long trains has been that the cost of controlling a train is practically the same whether it be long or short, but train despatchers as well as train crews have much more trouble getting very long trains over the road in time.

Excessive length of trains may and does interfere with their prompt and efficient handling at terminals or passing sidings, causing delay to other trains.

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* *Engineering News*, December 7th, 1911.

† *Engineering News*, July 18th, 1912.

Heavy locomotives and high speed increase cost of maintenance very rapidly as they pass the normal, 125 lb. rails of open-hearth steel, with special alloys, are already being considered to meet the demands of heavy traffic, and this expense is a charge against the heavy locomotive.

Probably the most important consideration of all, however, is the item of rolling stock repairs which has increased very rapidly with the increased weight of train and power of the locomotive. A hauling power of 25 tons and upward cannot be transmitted through a long freight train without exerting enormous stresses upon draft gear, couplers, car sills, floors, etc. It is not the mere push or pull of the locomotive, but the transmission of the power in a series of waves or shocks, especially on lines with broken profiles.

This matter of the proper design of rolling stock, and the strengthening of the underframes and draft rigging, is one which is receiving considerable attention, but, of course, the raising of the general standard of efficiency in a matter of this kind is very gradual, and in the meantime there is a temporary halt.

Enough has been said, however, to show that the 5 ft. 6-in. gauge offers no advantage in allowing the use of heavier cars or trains than are now in general use or in contemplation on the 4 ft. 8½-in., and the latter is capable of much further expansion along these lines, even beyond what are to-day considered to be the economic limits.

Train Resistance.—The term "train resistance" is used to denote the combination of forces which have to be overcome to produce the movement of the train, these forces being affected by the character of the road and vehicles and, therefore, to some extent, by the gauge.

Train resistance is of two kinds: that due to the internal resistance of the train, and that due to gradients and curvature. Numerous experiments have been made to determine the amount of the former, with varying results due to the complex nature of the problem, but one fact is firmly established, that the resistance per ton of train is much less for loaded cars than for empties, and less for heavier cars than for light ones. For a train of loaded 50-ton coal or mineral cars, all of the same type, the resistance, at ordinary freight-train speeds on straight, level track, is often as low as 3 lb. per ton, whereas trains of miscellaneous cars in the United States, including a fair proportion of cars as light as 30 tons and some empties, will have a resistance as

high as from 7 to 10 lb. per ton, 7 lb., however, being about the maximum with fairly good conditions. There is no doubt, therefore, that in this respect the narrow-gauge, with its much lesser average weight per axle, is at a disadvantage when compared with the broad-gauge, but just how much it is impossible to say.

The following table shows the effect of the capacity of cars on the loads which a consolidation engine (118 tons) can haul on a 0.5% grade at 20 miles per hour. The capacity includes weight of car and load, and the tonnage is based on the formula of the American Railway Engineering Association ($R = 2.22\ T + 121.6\ C$):

20-ton	cars	.....	1 570	tons.
30	"	"	1 775	"
40	"	"	1 910	"
50	"	"	2 010	"
60	"	"	2 090	"
72	"	"	2 160	"

The resistance due to grade is the force required to lift a certain weight a certain height; its computation is an exact mathematical proposition, and is not influenced in any way by gauge.

The resistance due to curvature is probably slightly less on narrow-gauge than on broad-gauge. It is of two kinds: one is the friction of the flanges of the wheel on the rails, which is probably in direct proportion to the force required to change the direction of a body tending to move in a straight line, this being a function of the speed and weight, and having nothing to do with the gauge; the other is the slipping of the treads of the wheels on the top of the rails, and this varies with the width of the gauge. That the latter is a comparatively unimportant factor is realized when one considers that, for example, on a curve of 400 ft. radius with a central angle of 90°, the difference in the length between the outer and inner rails for 4 ft. 8½-in. gauge is 7.39 ft. and for the meter-gauge, 5.15 ft. On a curve of 1 000 ft. radius with a central angle of 30°, which is about an average curve in the Argentine, the difference between the outer and inner rails would be:

Broad	.....	2.88 ft.
Medium	.....	2.47 "
Narrow	.....	1.72 "

When it is considered that about 90.4% of the alignment in the Argentine, including in this all the mountain lines, is on tangent, this additional resistance due to the narrower gauge is practically negligible. On a strictly mountain line, with very heavy and continuous curvature, this additional resistance might be considered perhaps to be equal, roughly, to the additional resistance of the smaller class of rolling stock, but, for the average line, it would be considerably less. On the other hand, locomotives for the narrow-gauge, of equal weight and power with those of wider gauge, must necessarily be longer, and, therefore, the resistance encountered by them in passing around curves would be somewhat greater than with the shorter engines for wider gauge.

Taken altogether, therefore, it seems quite safe to say that the resistance per ton of train on narrow-gauge lines is greater than on wider gauge, and the cost of operation is thereby increased, though it is hardly possible to calculate the amount.

Costs of Construction.

The reasons which are most generally advanced by the advocates of narrow-gauge for the supposed economy in first cost of construction are as follows:

1.—The use of sharper curvature is supposed to permit better adaptation of the line to the conformation of the ground.

2.—The cost of earthwork is supposed to be less:

a.—For reason No. 1.

b.—Because they are narrower.

3.—The cost of bridging is supposed to be less:

a.—Because they are to carry lighter loads.

b.—Because they are narrower, especially the masonry.

4.—The track and rolling stock are supposed to be lighter.

All these reasons are invalid except those directly affected by the width, that is to say, the width of earthwork and the length of ties; and, theoretically, it is questionable if even these are. If the bridges, track, and rolling stock are lighter, they are correspondingly less efficient, and, generally, in lesser proportion, as, for instance, a 30-lb. rail will only carry one-quarter the load that a 60-lb. rail will.

In nearly all the discussions, etc., in regard to gauge, comparisons of costs are made between two or more lines already built, or the average cost of all the broad-gauge lines of a country is compared with the average cost of all the narrow-gauge lines. Such comparisons, of course, are valueless for the determination of the effect of gauge on cost of construction. The broad-gauge lines usually are of a much heavier type of construction, have heavier rails, heavier rolling stock, and more of it, expensive terminals, etc., as they usually do more business. To give only one example of the many which have been found in looking over previous discussions, the following,* is a very fair example of the kind of argument used and information on which arguments have often been based. Speaking of Indian railways, the statement was made that broad-gauge costs 2½ times as much as meter-gauge. If the Government of India had pinned its faith to broad-gauge it would now (1906) have 7 000 miles less of railroad. The cost of line in India having been £11 775 per mile for broad-gauge, as against £4 700 per mile for meter-gauge.

In the discussion following, it was pointed out that the broad-gauge lines did more business, had heavier work, that is, greater quantities of earthwork, larger terminals, etc., and that to take only one item, that of bridging and tunnels alone, these had cost Rs. 10 792 ($3 500) per mile for the broad-gauge as against Rs. 2 149 ($700) for the meter-gauge, so that if the meter-gauge lines had been built in the same location as the existing broad-gauge and equipped to do the same business (if this were possible) they would have cost quite as much.

In connection with the further discussion of this paper, the following estimates were made of the difference in cost of building broad- and narrow-gauge lines in India:

Sir John Hawkshaw	£360	per mile	= $1 800
Sir John Fowler	633	" "	= 3 165
Sir George Bruce	200	" "	= 1 000
Sir Guilford Molesworth	425	" "	= 2 125
and in Victoria, Australia	261	" "	= 1 305
and South Australia	350	" "	= 1 750

It is to be noted that these differences are somewhat less than those shown in the estimates which follow, and are more nearly what they

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV.

might be if the calculations were made on a strictly theoretical basis, and including only the actual outlay for labor and material. In the estimates which follow (which are discussed in detail), not only has every reasonable allowance been made for any difference there is, but the same proportion is carried through the percentage of contingencies, interest during construction, etc., which tends to increase them, so that what difference they show is the maximum possible.

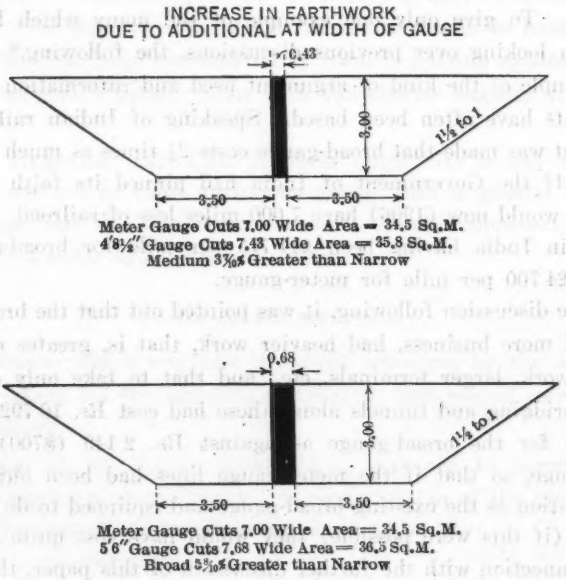

FIG. 4.

The only satisfactory method of determining the effect of gauge on cost of construction is to calculate what effect a difference of gauge would have, assuming the same conditions.

We have already seen that, within the limits of practical railroading for the gauges under consideration, curvature and gradients are not affected by gauge, therefore the location would be the same for lines of either gauge through the same country, for, in order to make a fair comparison, we must assume the road designed in either case to do the same business.

Taking each item of cost, therefore, we find:

Right of Way, Station Grounds, Terminals, etc.—These are not affected, the difference in width of earthwork being only 0.70 m. between meter- and broad-gauge, so that the width of right of way is not changed. If the tracks are spaced closer in terminals for narrow-gauge, they must be longer to hold the same tonnage.

Clearing and Grubbing.—There is no difference.

Earthwork.—This item is the one most affected by gauge, but the proportion is very small, as will be seen by the diagram, Fig. 4, showing, in the case of a cut or fill 3 m. deep, 4% more for medium over meter and 6% more for broad over meter. In making the calculations in the following tables, for lines with very heavy earthworks, an additional allowance has been made to cover the extra width in mountain work with steep transverse slopes, where the additional width involves somewhat more earthwork.

Bridging and Culverts.—The width of masonry is increased somewhat by reason of increased width of banks, but the amount is very small, about 2 to 4% for abutments, and from 0.2 to 1% for culverts, the end walls of the latter being the same, with only an increase of from 1 to 2 ft. in the length of the barrel. The steel work will be the same.

The latter assertion may be questioned, in view of the fact that much narrow-gauge rolling stock is lighter than medium or broad-gauge, but in order to avoid too much complication, it is assumed for purposes of this comparison that the same number of trains will handle the business, therefore the same weight of engine will be assumed. If any other assumption is made, that is, that heavier engines are used for the wider gauges, that means less trains and less cost of operation, which would compensate for the additional weight of the steel. The total cost of the actual bridging, omitting abutments, etc., is such a small proportion of the total cost of the railroad that any slight difference in this item would be very small.

Track.—Rails and fastenings will not be affected. The cost of rails is taken at $30 at Buenos Aires with about $150 per mile added for freight. The length of the cross-ties is affected directly as the width of the gauge, but as the function of the sleepers is to distribute the load, as well as to hold the rails in position, there would only be as many required for broad-gauge as would give the same bearing

area on the soil or ballast as those of the narrow-gauge, and, therefore, theoretically, there would be no difference in this item. Practically there is, however, but with the result that a better track is obtained. The number of the sleepers has been assumed, therefore, at 1 400, 1 500, and 1 600 per kilometer (2 254, 2 415, and 2 576 per mile) for broad-, medium-, and narrow-gauge, respectively.

Quebracho ties are used almost exclusively on the Argentine railways; they are very hard and durable, and, owing to their high cost, are spaced rather farther apart than is usual in the United States. The average price, including freight, at Santa Fé, a fairly central point, for the area under consideration, is:

5 ft. 6-in. gauge	$1.90	each	=	$2 660	per	km.	$4 283	per mile.
4 ft. 8½-in. gauge	1.60	"	=	2 400	"	"	3 864	" "
Meter-gauge	1.20	"	=	1 920	"	"	3 091	" "

Laying and Surfacing.—This has been assumed to be somewhat more for medium- or broad-gauge, on account of the greater width, but, although it is a little doubtful whether this is really warranted, it is taken in order to be on the safe side.

Ballasting.—This is assumed to be the same for all gauges, for, in order to carry the same loads, a greater depth of ballast is required for narrow-gauge, where the loads are concentrated, than for the wider gauge. Then, again, the impact is greater on narrow-gauge, due to the greater height in proportion to the width of the rolling stock.

Yards and Sidings.—These are taken at a percentage of the cost of the track in each case. Here again, theoretically at least, there should be no increase in cost due to gauge, for, although the broad-gauge track costs more per linear foot, there is not so much of it required, broad-gauge trains of the same net tonnage requiring a lesser length of track than narrow-gauge trains. As, however, on lines of comparatively light or medium traffic the yards and sidings are likely to be made the same length in any case, the increase is made as shown.

Cattle Guards.—These will be slightly wider for wider gauge.

Miscellaneous Structures, etc.—Fences, Track Signs, Buildings of all kinds, Water Supply, Telegraphs, Docks and Wharves, General Expenses of Construction, Engineering and Legal Expenses, would not be changed.

Contingencies, Expenses of Promotion, Interest During Construction.—These are figured at percentages of the totals.

Equipment.—For the same amount of traffic, the equipment would probably cost slightly less for the wider gauge on account of the larger capacity of the cars, but, assuming as nearly as possible the same weight of engines, etc., this would not be a great deal.

Tables 15, 16 and 17 show the differences due to gauge alone on lines of light, medium, and heavy earthwork and bridging, and assuming the same traffic and carrying capacity on all. It will be noted that for lines of light and medium construction there is very little difference in the percentage of increase, what difference there is being a less percentage where the work is heavier, medium-gauge averaging 6% more than narrow-gauge and broad-gauge 10% more than narrow. For very heavy lines, that is, mountain lines with steep transverse slopes, there should be some additional increase in earthwork due to the increased width. This has been allowed for, and in this class of lines the medium-gauge might be assumed to cost 11% more and the broad 16% more than the narrow.

The summary of the totals is given in Table 14, in which the percentage is the increase above the cost of narrow-gauge.

TABLE 14.—SUMMARY OF COSTS.

Gauge.	COST PER MILE, IN GOLD DOLLARS.			COST PER KILOMETER, IN GOLD DOLLARS.		
	Light.	Medium.	Heavy.	Light.	Medium.	Heavy.
Narrow.....	$29 772	$54 760	$92 573	$18 458	$33 951	$57 395
Medium.....	31 581 + 6%	57 920 + 6%	102 892 + 11%	19 580	35 910	63 793
Broad.....	32 861 + 10%	60 430 + 10%	107 553 + 16%	20 374	37 467	66 683

It may be noted that the assumed increases in Table 14, which the writer believes to be the maximum or outside amounts, are not more than the usual allowances for contingencies in work of this magnitude.

COST OF CONVERSION.

The following estimate of the cost of changing the gauge, although not based on detailed surveys or calculation of quantities, is believed to represent average conditions, based on observations by the writer after an inspection of practically all the meter and standard-gauge

lines of the Argentine, as well as of a considerable proportion of the broad-gauge lines.

It is believed that it can be safely assumed that the cost of conversion, including a liberal allowance for rolling stock, will not exceed the figures given, and will probably be less. It is to be noted, however, that these estimates do not include any betterments, such as heavier rails, renewing bridges, etc., or any other works of improvement which it might be desirable to carry out at the time the gauge would be changed; but they do include an allowance for all new sleepers. It may also be noted that most of the lines which would be changed first are through easy country, and the cost would be less than the average.

TABLE 15.—Cost of New Construction.

Light Earthwork and Bridging.

	Cost per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½ in.	5 ft. 6 in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	200	200	200
Earthwork	4 000	4 310	4 570
Bridges	2 000	2 150	2 300
Culverts	2 000	2 150	2 300
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply etc.	2 000	2 000	2 000
Telegraph and telephone	500	500	500
Docks and wharves	50	50	50
General expenses and administration	300	300	300
Engineering	1 000	1 000	1 000
Legal expenses	250	250	250
	$23 280	$24 940	$26 140
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% — 15%	3 492	3 741	3 921
	$26 772	$28 681	$30 061
Equipment (rolling stock, etc.) assumed for same amount of traffic	3 000	2 900	2 800
	$29 772	$31 581	$32 861
Per kilometer	$18 458	$19 580	$20 374

TABLE 16.—Cost of New Construction.
Medium Earthwork and Bridging.

	Cost Per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½ in.	5 ft. 6 in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	250	250	250
Earthwork	18 450	19 430	20 260
Bridges	5 000	5 500	6 000
Culverts	5 000	5 300	5 600
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 080
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc.	2 500	2 500	2 500
Telegraph and telephone	550	550	550
Docks and wharves	50	50	50
General expenses and administration	400	400	400
Engineering	1 300	1 300	1 300
Legal expenses	270	270	270
	$44 750	$47 580	$49 850
Contingencies, 5%, Expenses of promotion, 2%, Interest during construction, 8% — 15%	6 710	7 140	7 480
	$51 460	$54 720	$57 330
Equipment (rolling stock, etc.) assumed for same amount of traffic	3 300	3 200	3 100
	$54 760	$57 920	$60 430
Per kilometer	$33 951	$35 910	$37 467

The following* shows the actual cost of changing certain lines in India from meter to 5 ft. 6 in. The costs include rolling stock and, in at least one case, the Salt Branch of the North Western, the cost of changing from 41-lb. to 75-lb. rails, and in the last case there were "heavy charges for bridge renewals and heavy permanent way".

Cost of Converting Meter-Gauge to 5 ft. 6-in. in India.

		Per mile.	Per kilometer.
Salt Branch of North Western,	50.0 miles	£2 867 = $14 350	= $8 600.
Nagpur-Chattisgarh,	145.2 "	£2 583 = $12 915	= 7 750.
Kotkapura-Firozur,	27.9 "	£1 472 = $73 60	= 4 415.
Gudur to Nellore,	24.2 "	£4 125 = $20 625	= 12 374.

* From *Minutes of Proceedings*, Inst. C. E., 1906.

TABLE 17.—Cost of New Construction.
Heavy Earthwork and Bridging.

	Cost Per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½ in.	5 ft. 6 in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	300	300	300
Earthwork	39 490	44 500	46 000
Bridges	10 000	12 000	13 000
Culverts	10 000	11 000	12 000
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 080
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc.	3 000	3 000	3 000
Telegraph and telephone	600	600	600
Docks and wharves	50	50	50
General expenses and administration	500	500	500
Engineering	1 800	1 800	1 800
Legal expenses	300	300	300
	$77 020	$86 080	$90 220
Contingencies, 5% Expenses of promotion, 2% Interest during construction, 8% } 15%	11 553	12 912	13 533
	$88 573	$98 992	$103 753
Equipment (rolling stock, etc.) assumed for same amount of traffic	4 000	3 900	3 800
	$92 573	$102 892	$107 553
Per kilometer	$57 395	$63 793	$66 683

The author of the paper, Sir Frederick Robert Upcott, who gave these costs, estimated that the cost of changing the existing meter-gauge lines of India to 5 ft. 6-in. would be £2 500 ($12 500) per mile, or about £1 500 ($7 500) per kilometer.

It is stated* that the cost of changing the gauge of the lines in Queensland and West Australia from 3 ft. 6-in. to 4 ft. 8½-in. will be $111 000 000, or an average of $16 600 per mile ($10 000 per km.) for the 6 662 miles to be changed.

It has also been stated that the estimated cost of changing the gauge of the railways in Japan will be some $125 000 000 for about

* *Railway Age Gazette*, July 11th, 1913.

5 000 miles, or $25 000 per mile ($15 000 per km.). This seems high, but, of course, the topography of the country is quite rugged and mountainous and the railways have many large bridges and tunnels.

TABLE 18.—Estimate of Cost of Changing 1 Mile of Meter-Gauge to Medium- or Broad-Gauge.

Assuming Average Conditions on the Meter-Gauge Lines of the Argentine.

		Gold Dollars.	
		Medium.	Broad.
Right of way, station grounds, terminals, etc., $10 000 for additional terminal land for every 300 miles		$33	$33
Earthwork: Say, 5% additional for medium and 7% for broad on assumed present average cost of about $20 000 per mile		1 000	1 400
Bridges and culverts: lengthening		2 000	2 500
Track: Medium gauge:			
2 400 new sleepers at $1.60	$3 840		
New fastenings	250		
Taking up, relaying and surfacing	1 500		
		5 590	.....
Broad-gauge:			
2 600 new sleepers at $1.90	$4 940		
New fastenings	250		
Taking up, relaying, and surfacing	1 500		
		.....	6 609
Yards and sidings: 20% of main line		1 118	1 338
Cattle guards, nominal		15	15
Fences, track signs, etc., no change		.....	.....
Buildings, shops, stations, section houses, turntables, water supply. Will only involve changing tracks and spacing in shops, turntables, engine-houses, and running sheds, say $200 000 for every 300 miles		630	700
Signals and interlocking		75	75
Telegraph and telephone, no change		.....	.....
General expenses and administration		100	100
Engineering		250	250
Legal expenses		25	25
Interest during construction		600	700
Contingencies		600	700
		$12 036	$14 526
New rolling stock, say $5 000 per mile.			
Credit old rolling stock 3 500 per mile		1 400	1 600
Per mile		$13 436	$16 126
Per kilometer		8 330	9 998

Cost of Operation.

Various statements have been made in regard to the relative costs of operation and maintenance of lines of different gauges, almost invariably however, so far as the writer has been able to discover, based on comparisons of the costs on two existing lines or groups of lines. Such comparisons are not of great value, for no matter how generally similar two lines may be, or seem to be, the inherent differences in

physical condition, amount and character of traffic, rolling stock, management, policy, etc., will be considerable. An attempt was made by the writer to analyze the details of the operating costs of certain lines, and from this to form some idea of the actual effect on each item of a change or difference in the gauge, but it was found that there was so little information available that the result would be too much in the nature of a pure guess to be of any value, and as there seems to be no really good way of determining what actual effect a difference of gauge would have on the operation and maintenance costs, such comparisons between the results on lines of different gauges in various parts of the world as are available are given for what they are worth, followed by some discussion of the effect of gauge on maintenance of permanent way and an analysis of the possible economies by the use of heavier rolling stock, etc., both in maintenance of equipment and costs of operation.

Indian Railways.—In regard to the railways of India, the following statement was recently made by Sir Guilford Molesworth, showing that although the train-mile costs on the narrow-gauge lines of that country were lower than on the broad, the net ton-mile costs were higher.

Taking the East India Railways (5 ft. 6-in.) as 100, the costs on the following narrow-gauge lines were:

	Per train-mile.	Per ton-mile.
Assam Railway	71	315
South Maharatta	56	280
Burma Line	64	290

A statement prepared by Sir Frederick Upcott, giving the comparative statistics of eight of the principal broad-gauge and eight of the principal narrow-gauge lines, for 1903, showed that, though the earnings on the narrow-gauge lines gave practically the same returns on the capital invested as on the broad-, the average charges were:

	Cents, in gold.	
	Broad.	Narrow.
Per passenger-mile	0.42	0.40
Per freight-mile	0.92	1.14

and the cost of carriage of freight per ton-mile was:

0.5 cent for the broad-gauge.
0.6 " " " narrow-gauge.

It is interesting to note, here, that the average weight of trains on the broad-gauge was 167 tons, as compared with 88 tons for the narrow-gauge.

Australian Railways.—The ton-mile costs in Australia are not available, but the train-mile costs for 1910-11 were as follows:

Broad	$1.20
Medium	1.09
Narrow	1.32

These figures, taken together with the fact that the net earnings per mile of the medium-gauge lines were double those of the narrow-gauge, indicate the inferiority of the earning power of the latter.

Argentine Railways.—In the Argentine the average operating costs of all lines for 1909 were as follows:

	Broad.	Medium.	Narrow.	Broad.	Medium.	Narrow.
	Per train-kilometer.			Per train-mile.		
Dollars, gold....	$1.006	$0.755	$0.876	$1.620	$1.216	$1.410
	Per ton-kilometer.			Per ton-mile.		
Cents, gold.....	0.928	0.957	0.967	1.494	1.540	1.557

The ton-mile (ton-kilometer) used for these figures is that of the "peso util" or "paying weight" transported, and includes passengers, with ordinary baggage, calculated at 100 kg. (220 lb.) each, excess baggage, and express, as well as freight; but, as the proportion of revenue from passengers, baggage, and express is much greater on the broad-gauge lines, being 31% of the total as compared with 19% for the narrow-gauge, the comparison is hardly fair to the broad-gauge. It shows, however, the same tendency to low cost per ton-mile for the wide-gauge, although the train-mile costs, due to the heavier trains hauled, are higher.

It is to be noted, as previously stated, that the medium-gauge lines, do not offer any real comparison on account of local conditions, and this applies also to the ton-mile costs on these lines, which are given later.

Taking the figures as they stand, and using the number of ton-kilometers for 1909, they show a saving in operating costs of a little more than $100, gold, per kilometer ($160 per mile) per annum in favor of the broad-gauge, which, of course, is not very large. In discussing these costs, however, consideration must be given to the essential difference and physical characteristics of the broad- and narrow-

gauge lines at this time (1909). The narrow-gauge lines of the Argentine were then doing a fair amount of business, as more or less local lines, that is, none of them reached the National Capital nor did they partake of the character of trunk lines. The Central Cordoba was then building to Buenos Aires, and now (1911) all the narrow-gauge lines realize that, if they are not to be entirely crowded out, they must get in close touch with the National Capital in order to stay in business, and spend large sums for capital account to put their lines in condition even to meet reasonably the competition existing or to be expected. The narrow-gauge lines were then capitalized at about $28 000, gold, per km., as compared with $42 000, gold, for the broad-gauge, but they are now faced with the necessity of capital expenditure which will bring them well up toward the capitalized value of the broad-gauge. A large part of the capital of the latter is invested in the expensive Buenos Aires terminals, and other facilities at all points on their lines, which they have found it necessary to have in order to do their business, which the narrow-gauge lines have yet to acquire and develop, and on the capital expenditure for which they must earn the interest. It is also quite certain that many of the narrow-gauge lines would have to be double-tracked in order to handle the business.

If it could be assumed that the foregoing figures would hold good for any amount of business to be hauled, they would not present a very formidable argument for the broad- as compared with the narrow-gauge, but all railroads, as well as all other enterprises in any country as progressive as the Argentine, must grow with the country or be crowded out, and this is what is beginning to happen to the narrow-gauge lines. The business coming to them is more than they can handle economically with the plant they have, because it tends to cause congestion.

The need of adequate facilities for doing business and the increased cost of operation due to their lack when the amount of business offered passes a certain limit, was shown in 1906 in the United States, when many of the railroads of the country suffered serious losses because of the congestion caused by the large amount of freight offered for transportation which they had not the facilities for handling.

The real test, of course, is the final net earnings on the investment, and here the broad-gauge lines, in spite of their heavier capitalization,

are a long way ahead, with average earnings of 5.66% on the capital invested, as compared with 2.41% earned on the capital invested in the narrow-gauge lines, so that it is evident that the ton-mile figures do not tell the whole story.

Taking freight alone, on the basis of these 1909 figures, the receipts per ton-mile make a still better showing in favor of the narrow-gauge, the averages being:

	Freight receipts.	
	Ton-kilometer.	Ton-mile.
Broad,	1.17	1.95 cents, gold.
Medium,	1.26	2.10 " "
Narrow,	1.05	1.75 " "
Average for all lines,	1.15	1.92 cents, gold.

It may be noted, also, that the rates on the Central Cordoba and Santa Fé lines are the lowest of any lines in the country, and it is these which tend to lower the average for the narrow-gauge lines, but it is believed that the explanation given above shows the reason for the apparent cheaper cost of operation, or, rather, lesser rates on these two narrow-gauge lines. It may also be added that the property of neither has been kept in first-class physical condition, and, as already stated, their net earnings show a much lower percentage of the capital invested than do the broad-gauge lines. The Government lines, also, which are narrow-gauge, are operated at a loss. If the freight charges on these lines were fixed to give the same return on the capital invested they would, of course, be much higher.

Comparison of Ton-Mile Costs.—The average receipts per ton-mile of the Argentine railways in 1909 are given above. The average for the United States in 1912 was 0.85 cents. On 70% of all the railroads the average was less than 1.0 cent per ton-mile, and the lowest average rate was 0.48 cent per ton-mile.

The actual cost of moving freight on the Bessemer and Lake Erie Railway in 1909 was stated to be* 0.230 cent per ton-mile; this included terminal charges on a road only 144 miles long; adding interest on the investment would bring the total cost to 0.280 cent per ton-mile.

Maintenance of Permanent Way.—In regard to maintenance, it seems quite evident that this would be considerably more on narrow-

* *Engineering News*, April 21st, 1910.

gauge lines having any large amount of traffic, and especially if it was desired to run passenger trains even at reasonably fast speeds.

If it were possible to build and maintain a track without inequalities, or if the inequalities of the line and the forces tending to produce oscillation could be reduced in the same proportion as the width of the gauge, the cost of maintenance might be considered as proportional to the width, but these conditions are unattainable.

The narrower the gauge the greater is the angle through which the vehicle is canted laterally through a certain depression or elevation of one of the rails, and the greater, therefore, is the inequality produced in the loads on the springs on opposite sides, and, consequently, on the two rails also, so that the narrower the gauge the greater the extent of lateral oscillation to which any given inequality in the line will give rise, and this is a point of especial importance in districts where, from the variations of climate or other influences, the permanent way is likely at times to get more or less out of repair.*

It is to be noted, also, that the use of stone ballast is almost indispensable on narrow-gauge lines, in order to have track on which speeds of more that 30 miles per hour are safe, and that the depth of the ballast must be increased in order to distribute properly the loads which, in the narrow-gauge, are concentrated on a smaller area.

A few years ago it was quite generally thought that the principal function of the ballast was to afford proper drainage for the track, though even as long ago as 1887-90 experiments, which were conducted in Germany by Railroad Director Schubert, had indicated the value and necessity of ballast for the proper distribution of the loads. In view of the great increase in axle loads in recent years, an extended series of experiments was made in 1908-10 by the Pennsylvania Railroad to obtain further light on this phase of the subject.† These experiments showed quite clearly and conclusively the part played by ballast in the distribution of the loads, and the necessity of a sufficient depth to take care of this. It is quite certain, therefore, that the concentration of loads on lines of narrow-gauge, both by reason of the smaller bearing area of the sleepers and the additional impact due to the higher center of gravity and greater oscillation, must be taken care

* See also p. 566 *et seq.*

† *Proceedings*, Am. Ry. Eng. Assoc., 1912.

of by an increase in depth of ballast, if equally good results are to be obtained.

Maintenance of Equipment.—It is quite difficult, if not impossible, to make any estimate of the effect of gauge on the cost of maintenance of equipment. It is quite sure that the cost of maintenance of both locomotives and wagons is not proportionate to either their size or capacity, that is to say, it does not cost twice as much per annum for the maintenance of a 200-ton engine as it does to maintain a 100-ton engine; two 25-ton cars will cost more to maintain than one 50-ton car doing the same work. The passenger equipment for narrow-gauge has only about three-quarters of the carrying capacity of the broad-gauge for vehicles of the same length, so that, to do the same amount of business, there will be a larger number of vehicles, larger proportion of dead weight per passenger, and, therefore, a larger cost of maintenance per revenue unit.

If it could be assumed that all or even the greater proportion of the business of the Argentine, if carried on broad-gauge lines, could be handled by locomotives and in freight cars of twice the capacity of those now in general use on the narrow-gauge, one could say at once that the cost of maintenance of equipment would be greatly lessened thereby. This, however, is not the case, and there are many locomotives and cars in use, and of a useful size, on the narrow-gauge, equal in capacity to many on the broad-, and it is only a certain proportion of the business that can be handled to better advantage in cars and by locomotives of larger capacity.

Taken altogether—passenger cars, freight cars, and locomotives—it seems quite sure than the cost of maintenance of equipment would be more on narrow-gauge lines than on the broad-gauge, by reason of the larger number of vehicles required to handle the same amount of business. Just how much this item would amount to it is not possible to say, though the figures in Table 19 may give some idea.

Effect of Larger Rolling Stock Units on Costs of General Repairs.—Assuming quite conservatively that half the business could be handled on the broad-gauge by equipment having 50% greater capacity than that now in use or possible on the narrow-gauge, then the number of units of rolling-stock would be reduced one-sixth. The cost of repairs on the larger units would be somewhat greater, but not propor-

TABLE 19.—Cost of Maintenance of Locomotives and Cars in 1909 on the Narrow-Gauge Lines of the Argentine.

	Number in service.	Repairs and Maintenance.	
		Total cost.	Cost, each.
Locomotives	500	$1 051 332	$2 102
Passenger coaches	599	258 477	430
Baggage cars	402		
Freight cars	13 540	683 175	47
Service cars	540		
		$1 992 984	

tionately so, say, roughly, only 50% of the increase. That is, for example, taking the case of the locomotives, 250 would remain as at present, each costing $2 100 a year for repairs, the other 250 would be replaced by two-thirds of the number, each unit having 50% greater capacity and costing 25% more for repairs, that is, there would be 167 each costing $2 625 for repairs per annum. The total cost of repairs would then be:

Locomotives	250 ×	$2 102 =	$252 500
	167 ×	2 625 =	438 375
Passenger coaches	300 ×	430 =	129 000
	200 ×	537 =	107 400
Freight cars, etc.	7 240 ×	47 =	340 280
	4 826 ×	59 =	284 734
			$1 825 289

This shows a saving of $167 695 per annum, equal to about $23 per km. of line, with the business of 1909. With a live policy of operation, the building of branches to develop the existing main lines, it is not unreasonable to assume that, by 1919, say in 10 years, the business will be increased to three times the present amount, and a large proportion of it will need to be handled in heavy trains.

There would be all the new mileage on which there would be a similar saving to that just shown, the new mileage would bring as much business to the old lines as these now have, and the natural increase in the country adjoining the old lines might be at least equal to the business now, so that on this item of repairs alone there would be a saving of from $70 to $100 per km. on the present lines, and at

least $20 to $30 on the new lines, or, say, an average of $50 per km. per annum on the whole system then in operation.

It is to be noted that if the change of gauge is brought to pass, the new equipment, if properly selected, would show a much smaller cost of repairs, as it would be better adapted to the requirements of the service. There would be also, without going to extremes, much of it of double the average capacity of the present narrow-gauge rolling stock, instead of only 50% greater as just assumed, so that the foregoing estimate is quite conservative.

Effect of Small or Large Cars on Train Resistance, Loading, Etc.— The effect of small cars on train resistance has already been pointed out.* Their effect on the cost of operation is indicated by the following:

Assume an engine of the Mikado (2-8-2) type:

Total weight of engine and tender..	246 600 lb.
Weight on drivers.....	110 500 "
Maximum tractive power.....	28 900 "

This is about as large an engine as can be built for meter-gauge of a type which will give large power capacity with ability to make fairly good speeds with heavy trains.

Engines of the Mallet or articulated type are built of much greater power, but, as that type only operates efficiently at very low speeds and is best adapted to sections of heavy gradients, it does not afford as good a basis for comparison for general conditions, in fairly easy country, similar to that found in the Argentine, where much of the freight must be moved quickly, as does the Mikado type.

The tonnage-rating formula of the American Railway Engineering Association is

$$R = 2.22\,T + 121.6\,C.$$

in which

R = Total resistance on level tangent, in pounds,
T = Total weight of train behind the tender, in tons,
C = Number of cars.

This assumes good, fair rolling stock and track, and that the resistance does not increase at ordinary freight train speeds between 7 and 35 miles per hour.†

* p. 590.
† *Proceedings*, Am. Ry. Eng. Assoc., 1910.

Using this formula and applying it to trains of fully-loaded cars on a line with grades of 0.6%, it is found that the engine assumed, when working to full capacity (*i. e.*, exerting the same power in each case), can haul the loads shown in Table 20.

TABLE 20.

	No. of cars.	Net tons per car.	Tare per car, in tons.	Gross tons.	Net tons of freight.
Broad-gauge cars	24	50	18	1 632	1 200
Standard-gauge cars	24	50	18	1 632	1 200
Meter-gauge cars	34	33	13	1 564	1 122

This is an increase of 78 net tons per train, as between meter- and standard-gauge. The formula deduced by Professor Schmidt, of the University of Illinois, from the results of extensive experiments made to determine the influence of car weight on train resistance, shows quite similar results, though even less favorable to the lighter cars.

The average receipts per ton of freight per kilometer in 1909 were 1.15 cents, gold. For the train under consideration, therefore, the average increased receipts for the same operating costs per train-kilometer would be 89 cents, gold, or, with an average goods traffic of one train daily each way, $1.78 per km. per day, or about $500, gold, per km. ($830 per mile) per annum.

It is stated that some of the traffic in the Argentine can be handled better in cars of small capacity. Assuming that half the business is handled in small cars on which there is no saving, there would still be an increased income of $250, gold, per km. per annum, due to the use of the larger cars.

Assuming, then, that an average kilometer of broad-gauge line costs from $3 000 to $5 000, gold, more to build than meter-gauge, the investment is warranted from this standpoint alone, and, taking the figures at the end of the paper, the change of gauge of the existing lines would also be warranted.

The foregoing takes no account of the further saving which may be effected on the wider gauge by the use of larger engines than are at all possible on meter-gauge, thereby further increasing the size and weight of trains and, consequently, obtaining lower operating costs, nor the fact that the speed of passenger trains is limited on the narrow-

gauge, both on account of the size of the engines and the lesser stability of trains.

Economy in Increased Train Loads by Reduction in Number of Trains.—The possibilities of the use of the larger cars and heavier engines depends, of course, to a large extent, on the kind of traffic to be handled. Such articles as cereals, cattle, timber and other forest products can all be handled to better advantage in large cars. The tonnage of these articles handled on the narrow- and medium-gauge lines of the Argentine in 1909 is shown in Table 21.

TABLE 21.

	Narrow.	Medium.
Cattle	219 631	232 680
Wool, hides, etc.	40 864	38 278
Cereals	955 448	294 659
Sugar	223 789	3 528
Minerals	35 500	.....
Construction material*	2 153 312	140 360
Firewood	768 828	30 045
Charcoal	302 881	33 495
	4 700 253	773 045

* Includes forest products.

That is, about 5 500 000 tons, out of a total of 8 500 000 tons, were handled on these lines, or 65% of the total. The average haul on all freight in 1909 was about 180 km., and the total length of lines about 9 000 km. It may be noted, also, that with the opening of the through connection to Paraguay and the extension of a railway line through Missiones, the shipment of timber suitable for building purposes from these northern points to the south is likely to assume large proportions, and this will all be long-haul business which must be handled cheaply to make it move.

The Mikado engine previously referred to can haul, say, 1 000 tons of net load on 0.6% grades. The largest engine yet built for narrow-gauge, the Mallet (articulated) engine previously referred to, might haul, say, 2 000 tons net load. There are, however, on some parts of these lines, especially in the southern section of the Central Cordoba and all through the Provinces of Entre Rios and Corrientes, grades of 1.0% or heavier. Taken altogether, therefore, considering the grades, the fact that the big articulated engine is not a useful type for

general use, and that a certain number of engines of the Mikado type must be kept in use for general freight services, it seems safe to assume that, even under the best of conditions on the narrow-gauge lines, the heavy trains will not exceed an average capacity of 1 500 net tons, whereas on the broad- or medium-gauge this could be doubled.

The operation of such extremely heavy trains, however, is not a practical proposition in the Argentine at present, nor is it likely to be in the immediate future, though the increasing distances to the new areas constantly being opened up will continually increase the tendency in this direction.

In referring to trains of this weight, it must not be assumed that the very much less average weight of trains has been lost sight of, the average net train loads in the Argentine in 1909 having been:

Broad	327	tons.
Medium	198	"
Narrow	211	"
All lines.....	293	"

The average train loads in the United States in 1910 for all lines was 380 tons, and in Group II, comprising the States of New York, Pennsylvania, and New Jersey, 502 tons, yet there are records of single trains of more than 8 000 tons gross, and the great reduction in the cost of transportation, which has been obtained in the United States in spite of the high cost of labor, has been brought about by the use of heavy trains for bulk traffic which can only be moved profitably at low rates.*

For the purpose of this argument, therefore, and to get some idea of the possible savings due to the reduction in the number of trains, it is entirely reasonable to assume that the practical train loads for the broad-gauge may easily be one-third greater than on the narrow-gauge, or say, 2 000 net tons, as compared with the possible maximum of 1 500 tons on the narrow-gauge.

Taking into consideration the growth of the country and a continuance of the railway development, it might be assumed that within a few years the amount of this class of traffic on the lines under consideration will be doubled to, say, 12 000 000 net tons. On the medium-gauge, with train loads of 2 000 net tons, this would mean 6 000 trains,

* See also p. 581.

and the narrow-gauge would require 8 000 trains. The average haul on this traffic is now 180 km., and the length of haul is practically certain to increase. An average engine division is about 150 km., and, considering the almost certain increased length of haul, one should add, say, one-third more to the number of trains (that is, train runs handled by one crew), making the numbers 8 000 and 10 667.

The present actual operating cost, that is, cost of coal, lubricants, train crews, repairs to rolling stock, etc., is about 50 cents, gold, per train-kilometer, of which about one-third is for fuel and water. This, however, is with trains very much lighter than those under consideration. With heavier trains the fuel and water would be increased, approximately, directly in proportion to the weight of the train. Repairs would be affected somewhat more, but not in proportion.

The average present train load to which these figures apply is about 500 tons. So taking this cost of 50 cents, of which, say, 15 cents is for fuel, water, and lubricants, and multiplying these items by three, and increasing the remainder by 25%, gives the following cost per kilometer per train of 1 500 tons:

Fuel, etc.....	15 cents	× 3	= 45	cents.
Other items..	35 "	+ 25%	= 44	"
Total.....			89	cents per train-kilometer.

In increasing the weight of the train from 1 500 to 2 000 tons, the other items would not be affected as much as by the increase as above from 500 to 1 500. Fuel might be increased 33%, though this is doubtful,* and the other items hardly at all, but, say, 10 per cent. We have, therefore, for a train of 200 tons:

Fuel, etc.....	45 cents	+ 33%	= 60	cents.
Other items...	44 "	+ 10%	= 48	"
Total.....			108	cents per train-kilometer.

Taking the average run as 150 km., the cost per train would then be:

For trains of 1 500 tons, 150 km. × 89 cents = $133.5 per train.
" " " 2 000 " 150 " × 108 " = 162.0 " "

and there would be:

On the narrow-gauge	10 667	trains at	$133.5	= $1 423 912
" " medium-gauge	8 000	" "	162.0	= 1 296 000
				$127 912

*See p. 584 *et seq.*

or equal to about $14.20 per km. of line, due to the reduction in the number of trains.

In the foregoing calculation it has been assumed that the costs of fuel, water, and lubricants increase directly in proportion to the weight of trains, but most of the larger modern engines have shown a marked economy in this respect, and the coal consumption, etc., has been quite a little less, proportionately, to the work done.

It may be noted that, in making these assumptions, it is not considered that there will be any additional costs on the standard-gauge, due to the operation of the heavier trains, which in some cases have to be considered. It is believed that a 1 500-ton train will be quite as destructive to the track on the narrow-gauge as a 2 000-ton train on the medium-gauge, and that the time factor will not enter into the question, as an engine designed to haul a 2 000-ton train on standard-gauge will make as good time and with less effort than will a narrow-gauge engine designed to haul 1 500 tons, on the same grades and alignment.

The Necessity of an Efficient Transportation Machine.—The possibility of the competition of other broad-gauge lines or of river transportation has been referred to, and it must be kept constantly in mind that cheap, but at the same time efficient, transportation is a necessity, both for the development of the country and on account of the river competition.

The following quotation from a recent review of the general railroad situation in the United States by M. C. Colson, an eminent French engineer, shows how cheap railroad service has affected transport by water, and has a direct bearing on the necessity of establishing a railway system in the Argentine, which will accomplish similar results there. It hardly seems possible with the narrow-gauge to approach the low cost of transporting bulk freight which can be achieved on the medium- or broad-gauge.

After pointing out that the mileage of railways in the United States amounts to 10% more than that of all the railways of Europe, though the population is only 90 000 000, as compared with 450 000 000 in Europe, he states:

"Before the railways [of the United States] became developed, attempts were made to extend the inland waterways by constructing many canals, but inland navigation gradually died out, except on the Great Lakes, where it has a quasi-maritime character, and on some

exceptionally placed waterways, as soon as a better sytem of transport became known. It was the railway which made it possible to develop with unexampled rapidity this great continent, to develop farms whose produce, sent to the Old World at very low prices, there produced the agricultural crisis of thirty years ago. Then it helped to extend a population there which will very soon consume all the produce of its own country, so that its growth is an important factor in the general rise in prices, which is now producing with us a crisis in an inverse sense of the former one, namely: the dearness of food. Finally, the railways, which carried coal and minerals at rates even lower than those charged for cereals, helped to give the United States an industrial development, the general growth and magnitude of which are just as surprising as the former development of farming."

Statistics are then given showing in detail the much lower costs of transportation in the United States as compared with those of Europe,* and he continues:

"In Europe 10-ton wagons are still the rule, 20-ton wagons are exceptional, and 40-ton wagons are very rare. In the United States, out of 2 400 000 wagons, there are only 12 000 which take less than 18 tons, and most can take 18 to 33 tons; 634 000 can take 33 tons, and 390 000 can take 42 to 54 tons. These working conditions, which have resulted in the historical development of American farming and industries, make it possible to have very low rates and still work at a profit."

SUMMARY.

The preceding discussion may be briefly summarized as follows:

Curvature.—Within the limits of the practical operation of railroads, a curvature of any radius which is feasible on meter-gauge is as practical on 4 ft. 8½ in., or even 5 ft. 6 in.

Gradients.—The working of trains on steep gradients is not affected by gauge, except in so far as the narrow-gauge cuts down the locomotive capacity, and is, therefore, at a great disadvantage.

Location.—In view of the foregoing, the narrow-gauge (3 ft. 0 in. to 3 ft. 6 in.) does not permit better adaptability of the alignment to the conformation of the ground.

Cost of Construction.—This is so little more for 4 ft. 8½ in. as to be more than counterbalanced by economy in operation.

Speed.—This is an important feature in railway operation which is adversely affected by narrow-gauge.

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* See also the statement on p. 581.

Stability.—The relative stability of narrow-gauge trains in motion is much less than that of trains on medium- or broad-gauge; consequently, high speeds are impractical and may be dangerous.

Track Stresses.—Much higher track stresses are produced on the narrow-gauge; there is greater impact and therefore more difficulty in maintaining track on narrow- than on broad-gauge.

Rolling Stock Cars.—The greater width of narrow-gauge cars in proportion to the gauge is considered a necessary evil rather than an advantage, and it is shown that there is probably no advantage to be gained by making the broad-gauge cars any wider than they are at present. In the matter of passenger cars, the narrow-gauge is at a decided disadvantage, both as to cost per unit of accommodation and in dead weight per passenger. In freight cars, there is little difference in cost, within the limits of size possible on the narrow-gauge, though cars of twice the capacity are available for medium- or broad-gauge. The increase in weight of modern freight trains, in order to decrease the cost of operation, has made heavier draft rigging necessary, thus tending to increase the proportion of dead to paying load to a much greater extent on small capacity cars than on the larger ones, than has been the case heretofore.

Locomotives.—The cost per unit of capacity of locomotives is about the same for each gauge up to the limit of the narrow-gauge, which limit for all types is about half of that of the 4 ft. 8½-in. or 5 ft. 6-in. There is an economy in the use of heavier types of locomotives not available for the narrow-gauge, and the use of cars of large capacity is a necessity for cheap transportation.

Train Resistance.—Train resistance is decreased by the use of larger units.

Capacity.—Figures are given tending to show that the relative capacity of narrow-gauge lines is not more than half that of medium- or broad-gauge.

Heavier Engines and Trains.—There is a general discussion showing the part played by increase of train loads in reducing costs in the United States.

Cost of Operation.—The ton-mile costs, on both Indian and Australian Railways, as well as those in the Argentine, are less on the medium- and broad-gauge than on the narrow-gauge, although the train-mile costs are often higher.

The lowest ton-mile costs on any railway in the Argentine are more than twice as high as any in the United States.

The saving in cost of operation, due to the use of larger cars and heavier trains, is considerable, and would in itself go a long way toward meeting the interest charge on the additional cost of the wider gauge.

Cost of Maintenance.—Maintenance of way probably costs more per traffic unit on narrow- than on medium- or broad-gauge on any but lines of very light traffic.

Maintenance of equipment should be less on medium- or broad-gauge by reason of the larger capacity, and, consequently, lesser number, of locomotives and vehicles, provided there is business to warrant their use.

Need of Efficient Transportation Machine.—The necessity of an efficient transportation machine to develop the country properly is pointed out, as well as the part played by the railways in developing the United States, the country most nearly comparable with Brazil and the Argentine.

The Most Suitable Gauge.

We now come to the consideration of the most suitable gauge for the development of these countries. Such details of the topography, general conditions, traffic, etc., as are necessary for a proper understanding of the requirements are given in Appendix B.

In the Argentine, as in India and Australia—the only other countries which have large areas, large traffic, and different gauges—whatever the cause may be, however different any two lines may be, it is a fact that the greatest development has been along the lines of the broad- or medium-gauge railroads, and the big business is done by them. This is shown in Table 22.

TABLE 22.—Receipts, in Gold Dollars, per Mile of Railway per Annum.

Country.	Year.	Gross Receipts.			Net Receipts.		
		Broad.	Medium.	Narrow.	Broad.	Medium.	Narrow.
Argentine	1911	$7 385	$3 450	$3 905	$2 340	$1 395	$1 000
India	1906	9 870	.....	3 910	4 845	.....	2 090
* Australia, Government	1911	6 985	8 135	3 515	2 565	3 165	1 190
Australia, Private	1911	2 040	3 850	2 510	1 000	3 055	1 150

*Leaving out South Australia, total receipts $26 700 for two gauges not divided; and Northern Territory, $1 875 narrow-gauge, too small to be averaged with others.

It is difficult to get close figures for Brazil, but for 1906—the latest for which official statistics are published—the three systems which contain among them the broad-gauge lines, namely, The Central of Brazil, The Paulista, and the São Paulo Railways, operate (counting both broad- and narrow-gauge) less than 15% of the total mileage of the country, and their receipts amount to nearly 65% of the total, averaging about $12 000 per mile of line.

Considering the actual experience of the other countries, therefore, and the preceding detailed discussion of the actual effect of gauge, there seems, to the writer, to be little opportunity for a difference of opinion that the 4 ft. 8½-in. gauge is the most suitable for almost all conditions, and that for new lines, even in, or perhaps more particularly in, mountainous districts, and where the amount of business to be expected is not large, the slight difference in cost is generally more than warranted by the possibilities of economy in operation. The question in the Argentine and Brazil, however, is whether the cost of changing the existing narrow-gauge lines is warranted, and, in the former country, whether the change should be to 4 ft. 8½ in. or to 5 ft. 6 in., and, by the adoption of this latter, unify the gauge of all the lines in the country.

The consideration of a change of gauge, if confined to the Argentine alone, is practically narrowed down to the question, whether it is possible to unify the gauges of all the railroads, or whether one gauge shall be eliminated, leaving only two instead of three? The following argument, as far as page 626, therefore, is confined to a consideration of the question as it affects the Argentine, and without reference to the adjoining countries.

The Best Gauge for the Argentine Alone.—The relative advantages of 4 ft. 8½ in. and 5 ft. 6 in. have been discussed at some length, and it is believed that it is shown fairly conclusively that 5 ft. 6 in. does not now (with the imposed restrictions as to loading gauge in the Argentine) offer any advantages, from a purely transportation standpoint, over 4 ft. 8½ in., nor would it be likely to, even if there were a possibility of taking full advantage of the extra width of gauge by designing rolling stock with the same proportionate overhang. It seems important to emphasize this latter statement, as it is generally supposed that the 5 ft. 6-in. gauge could be better utilized if full advantage were taken of the extra width, but this is not the case,

or, at least, only in a minor degree, and, if the 5 ft. 6-in. gauge is adopted, it will not be for any advantage it has over 4 ft. 8½-in., but solely in order to unify the gauges of all the lines in the Argentine.

It is a safe assertion that it is impracticable to work more than two different gauges, inasmuch as all lines must have access to the ocean ports, and although it is practical to work two gauges, by having a third rail, on the port railways, it is not practical to work three, and it is impractical to reserve a special section of a port for a special gauge.

The broad-gauge is undoubtedly there to stay, as it has the greatest mileage, the companies working it are by far the strongest financially in the country, and, to a large extent, are all controlled by mutually friendly interests which probably would not even consider a proposition to narrow the gauge.

The meter-gauge has the next largest mileage, nearly four times that of the medium-gauge, and it would be far more costly to convert the meter-gauge lines to medium- than *vice versa*. It seems, therefore, as if, between these two—the meter- and the medium-gauge—the medium might have to go. This would be particularly unfortunate, as it would be undoubtedly a long step backward, but there are many indications that a sentiment is rapidly growing against the existence of the three gauges.

The Government lines, all meter-gauge (except those in Patagonia which are 5 ft.-6 in.), have been built with the primary object of bringing the outlying sections of the country and the capitals of the Northern Provinces in touch with the National Capital, for both political and military reasons. They reach every provincial capital, except Entre Rios, Corrientes, Mendoza, and Buenos Aires, and are connected with the latter, the National Capital, by two separate lines of meter-gauge, the Central Cordoba and Compañia General de Buenos Aires, so that every capital (except Mendoza) and practically the whole of the northern end of the Chilian frontier and the frontier of Bolivia can be reached by through trains from the capital without change of coaches.

The frontiers of Paraguay, Brazil, and Uruguay, where they join the Argentine, can only be reached by the medium-gauge, and, though this gives access to them from the National Capital itself, if it becomes necessary to send troops to any of these frontiers, all, except those from the City of Buenos Aires and vicinity, would have to be transferred from cars of a different gauge. This, though it may appear to

be a far-fetched argument for changing the gauge of the Entre Rios and North East Argentine Railways to narrow-gauge, at least partly explains the growing sentiment that the Government should be able to reach these frontiers over lines of the same gauge which now enables it to reach every other part of the country except Bahia Blanca, and access to this latter by a meter-gauge line is only a question of time, if the rest of the meter-gauge lines remain as they are. The experience of Japan (see page 639), however, shows the inferiority of the narrow-gauge for military purposes.

There is also the feeling of the people of these two provinces, Entre Rios and Corrientes, that they are isolated. So long as their railroads stopped at the edge of the rivers, and they were obliged to leave the train and cross the water in some kind of a boat, they accepted that as inevitable. The opening up of the Entre Rios car-ferry route to Buenos Aires, however, has completely changed the aspect of the province, and has shown its inhabitants the great advantage of through railway communication, so that they will not be satisfied until they have the same kind of through communications with the great northern section of the country, Tucuman, Bolivia, and the Chaco, by car-ferry transfers between Parana and Santa Fé and between Corrientes and Resistencia.

From Santa Fé there are narrow-gauge lines reaching practically the whole country except the Cities of Bahia Blanca and Mendoza. It is extremely natural, therefore, that it should be proposed, as it is quite seriously, to cover the whole of the Provinces of Entre Rios, Corrientes, and Missiones with a network of narrow-gauge lines connecting at Parana and Santa Fé with the narrow-gauge net on the other side, or to have the Government take over the existing line and change the gauge to meter.

It seems to the writer, however, that these proposals are made without an adequate realization of what an efficient transportation machine is, and what it means to the future development of the country. The need of fairly rapid transportation for passengers, the character of the freight traffic, etc., are indicated in Appendix B.

The principal defect of the narrow-gauge, which has been already pointed out, is its much lesser stability and greater stresses, both to rolling stock and track, caused by the unevenness of the latter, combined with the much greater height in proportion to the width of support of the former. This makes stone ballast—a very expensive article in

the Argentine—an absolute necessity for the narrow-gauge, whereas the broad- and medium-gauge, in many cases, can handle the traffic fairly comfortably without it.

In considering the merits of the two gauges, there seems to be little need, at this late day, to prove that the narrow-gauge, as a transportation machine, that is, regarded solely from the standpoint of a machine for the safe, comfortable, rapid, and economical transportation of passengers and freight, is inferior to medium or broad. Even the most ardent advocates of narrow-gauge admit the inferiority, but claim that this is compensated by the lower first cost, and also, sometimes, that narrow-gauge lines are cheaper to operate.

The statement that narrow-gauge lines are cheaper to build is fallacious, but still persists in some quarters, because many narrow-gauge lines actually are built more cheaply than some medium- or broad-gauge lines. This, however, is not because they are narrow-gauge—at least not to any great extent, as has already been shown in detail—but because they are built of cheaper materials. They have lighter or lesser capacity. For the majority of the lines to be built in the Argentine, which will cost mostly less than $25 000 per km., the costs would be, approximately, for the same type of construction and including all interest and overhead charges:

Narrow-gauge, per kilometer	.....	$25 000,	gold.
Medium-gauge, " "	.....	27 500	"
Broad-gauge, " "	.....	27 750	"

There will be many hundreds of kilometers costing not more than $15 000, on which the difference would be even less.

The mistake most often made in discussing this question is in confusing the terms "Narrow-gauge railway" and "Light railway," though, of course, a "Light railway" may be of any gauge.

In some of the discussions on this phase of the subject, it has been claimed that if standard- or broad-gauge roads or branches were built with light rails, bridges, etc., it would not be possible to use the same rolling stock on them that is used on the main line, or lines of heavier construction, but this can hardly be supported. It must not be forgotten that the force of impact due to speed is a most important factor, governing the necessary strength of the track and other structures of a railroad, and though one would not expect to run the heaviest type of

locomotives over a light line, it is perfectly feasible to carry any rolling stock, except the heaviest locomotives, at low speeds on rails as light as 40 lb., and there is no economy in building bridges for any railway traffic that will not carry freight cars of 50 tons capacity running at speeds of 20 miles per hour, which is quite fast enough for branches of light traffic.

In regard to cost of operation, there is room for difference of opinion, as it seems to be impossible to offer proof of the actual effect of gauge on this. The matter has been discussed in some detail, and there seems to be no reasonable doubt that, assuming equal conditions, the cost would be less on the broad-gauge than on the narrow-, but, just how much, it would be difficult to say. Certain of the figures seem to show that the saving made possible by the use of larger rolling stock might alone be more than enough to compensate for the cost of conversion.

The problem, therefore, so far as only the Argentine is concerned, seems to be narrowed down to the question of either the unification of the gauge by conversion of all lines to 5 ft. 6 in., or of changing all the narrow-gauge lines to medium, or all the medium to narrow, and in any case the scheme adopted must embrace all the lines of the gauge to be changed. So far as having an efficient transportation machine is concerned, the problem would be solved by the conversion to the medium-gauge, but if this is to be considered seriously, the question naturally arises would it not be throwing away a great opportunity to go farther and convert all to 5 ft. 6 in., and so unify the gauge of all the lines in the Argentine (except certain of the mountain lines of light traffic which could remain as narrow-gauge feeders, at least for some time)? The additional cost of conversion to the broad-gauge instead of the medium would not be so very great, in comparison with the whole cost in a scheme so large as this, and it would not be unreasonable to assume that, to obtain a result of such far-reaching benefit to the country, the Government of the Argentine, like that of Australia, would assume a large share of the burden.

Although there can be little room to question the statement that, for new lines in a country where construction is as light as it is in the Argentine, the additional extra cost for the wider gauge is so small as not to be worth considering, when compared with its great advantages over the narrow, the important question is the justification,

from a financial standpoint, of spending the amount necessary for the conversion of the existing lines, which, up to the present at least, have not shown such great deficiency in handling their business as would seem to warrant the expenditure of this sum in improvements.

It must not be forgotten, however, that, as pointed out on page 604 and in Appendix B, the character of the narrow-gauge lines is just now undergoing a radical change, from that of more or less small local lines, doing a small local business without competition, to that of a large trunk-line system, many of the lines of which are or will be competitive either with other broad-gauge lines or with river transportation, and they must either develop into an efficient transportation machine, capable of handling such business as may be offered at rates and in a manner which will compare favorably with those elsewhere, or be eventually crowded to the wall by transportation systems which can and will do this.

The underframes and draft rigging of the larger part of the rolling stock of the narrow-gauge lines is so light as to prevent its use in trains of any weight or length. The strengthening of these, or the provision of adequate strength in new equipment, will make the dead weight relatively large in proportion to the paying load, and in every way the narrow-gauge is forever limited in the matter of speed, comfort, stability, and economy of operation when compared with the standard-gauge.

If no great development or extension of the existing system of railroads were to be expected, it is doubtful if the expenditure necessary for conversion would be justified, but if the future development of the country is to be on the scale which a knowledge of it and a study of the general condition of the world's food market seem to indicate, it is believed that it can be.

The possibilities of double-tracking the existing narrow-gauge lines instead of widening the gauge, must, of course, be considered, especially as the cost of this on the lighter lines would be little, if any, more than the cost of conversion, and double-tracking could be carried out as required and without disturbing traffic in any way. It is doubtful, however, if a double-track narrow-gauge line has much greater efficiency or even capacity than an efficiently managed single-track line of standard-gauge (12 000 000 tons per mile of line per annum, or a total of 650 000 000 tons per annum, are handled on the Bald Eagle Branch, a single-track line of the Pennsylvania Railroad, 54 miles in length),

and then the cost of maintenance of double track is practically double that of single track. (See also page 580.) The cost of maintenance per ton-mile for any volume of traffic which might reasonably be expected in any part of the Argentine, except in the vicinity of the National Capital, would probably be almost, if not quite, twice as much for a double-track narrow-gauge as for a single-track wide-gauge, and the possibly somewhat greater ease of handling trains would be more than offset by the other operating economies possible on the wider gauge. In mountainous or even rough country, of course, the cost of double-tracking would far exceed the cost of changing the gauge, and it has been shown that the narrow-gauge offers very little advantage in decreased original first cost in rough country.

Looking at the question from the broad economic standpoint of the future development of the Argentine, it seems to the writer that the policy of continuing the construction of narrow-gauge lines is wrong, especially as it is believed that it can be shown that the additional cost of an efficient transportation machine, which the narrow-gauge is not, can be fully justified from the financial point of view, as well as from the point of view of its absolute necessity for the proper economic development of the country.

Cost of Unification for Argentine Railways Alone.—In calculating the cost, one must consider, not only the cost of changing the existing lines, but looking farther ahead, estimate the additional cost of the completed system as broad-gauge, over and above what it would cost to complete it as narrow-gauge. That is to say, supposing there are 10 000 km. of meter-gauge to be changed and 10 000 km. more to be built; the actual cost of changing might be $8 000 per km., the cost of building as 4 ft. 8½ in., possibly $1 000 or $2 000 per km. more than if built as meter-gauge, so that the added cost per kilometer for the whole of the completed system would be about $5 000 per km.

Taking the country only as far north as 22° South Latitude (the southern border of Bolivia), there is, north of the line of the existing railway from Sante Fé to Tucuman, a region comprising some 100 000 sq. miles (260 000 sq. km.) in the Argentine alone, and say, roughly, 25 000 sq. miles (65 000 sq. km.) more in Paraguay west of the Paraguay River at present served by only about 1 250 km. of railway. In other parts of the Argentine the length of railways in comparison with the area (for 1909) is approximately as shown in Table 23.

TABLE 23.

	Per 100 sq. miles.
Federal District	152.0 km. = 94.0 miles.
Province of Buenos Aires	7.2 " = 4.5 "
" " Cordoba	5.0 " = 3.1 "
" " Santa Fé	8.0 " = 5.0 "
" " Entre Rios	3.1 " = 2.4 "
Similar units for other areas are:*	
United States	14.1 " = 8.75 "
France	27.8 " = 17.22 "
Prussian-Hesse	40.0 " = 24.08 "
Great Britain	52.6 " = 32.65 "

* *Bulletin.* Bureau of Railway Economics.

It seems reasonable to assume that to develop this northern region there should be at least 5 km. of railway per 100 sq. miles, or a total of 6 250 km., of which there is now in existence some 1 250 km., leaving 5 000 km. yet to be built.

There is in existence at the present time in the whole country about 9 000 km. of narrow-gauge railway, of which 1 000 km. are strictly mountain lines of light traffic which need not be converted for some little time. In the next 20 years there will be required, say 4 000 km. of extensions to the existing lines, outside of this northern zone. So that, taken altogether, mountain lines and all, there will be 9 000 km. of the narrow-gauge to be converted and, say, 9 000 km. of new lines to be built.

To complete the unification of the gauge to 5 ft. 6 in., the standard-gauge lines must be included, of which there will be about 2 500 km. to be converted and about as much more new line required for extensions. The total additional cost of the final broad-gauge system, as compared with narrow-gauge, would then be:

Conversion of 9 000 km. of meter-gauge to broad, at $10 000 per km.	$90 000 000
Extra cost of building, say 9 000 km. of broad-gauge instead of meter, at $2 500.	22 500 000
Cost of changing 2 500 km. of medium-gauge to broad, at $6 500 per km.	16 250 000
Extra cost of building 2 500 km. of broad-gauge instead of medium, at $1 500.	375 000
	$129 125 000

That is looking ahead, say, 20 years, and not considering the existing broad-gauge lines. The average cost per kilometer of line then in existence as broad-gauge would have cost about $5 000 per km. more than if the existing narrow- and medium-gauges are perpetuated.

The interest on this, at 6%, is approximately $300 per km. per annum, so that, taking into consideration all the figures given in the previous discussions, and the several other undeniable advantages pointed out, it seems entirely reasonable to expect that, even with only a fair amount of business, the earning capacity of the broad- over the narrow-gauge would justify this expenditure, and, in addition to this, it is believed that there are many other reasons, particularly the narrow margin of safety on the narrow-gauge, the necessity of keeping the track in almost perfect condition in order to insure the possibility of even reasonable speeds and a fair amount of comfort, which warrant the expenditure.

The total expenditure indicated in the foregoing makes full allowance for the losses, if any, which might be due to the change of rolling stock. If, as has been proposed, the change was spread over, say, 10 or 12 years, quite a large proportion of the existing rolling stock would be used up during that period, and at the end of that time, in all probability, there would be direct connection with the narrow-gauge lines of the West Coast and the remainder of the rolling stock could be disposed of to roads in those countries, thus greatly minimizing this item; also, the difference in the value of the old rolling stock and the new would be much more than counterbalanced by the economies possible in the operation of the new rolling stock, all of standard types and of similar classes, and all selected in the light of the present knowledge of the requirements of the conditions to be met and the wide range of types now available to meet them.

The Best Gauge for South America.—The foregoing argument is based entirely on the point of view of the Argentine alone, without considering any of the adjoining countries or the necessities of international communications. If the whole of that part of South America referred to, that is, the part shown on the map, Plate I, is considered, however, the solution of the question as to which of the two wider gauges to adopt, 5 ft. 6 in. or 4 ft. 8½ in., is not quite so apparent.

The adoption of the 5 ft. 6-in. gauge in the Argentine leaves Paraguay and Uruguay with the medium (4 ft. 8½ in.), and it can hardly

be expected that either of these countries, more especially the latter, with some 2 290 km. of line, would consider a change from which it could expect no possible benefit. Brazil, because of the existence of a small quantity of 5 ft. 3-in. gauge, might possibly be inclined to favor the 5 ft. 6 in., but there is not enough of it really to influence the question either way. The 5 ft. 3-in. is neither one thing nor the other, and it would be practically as easy to change it to 4 ft. 8½-in. as to 5 ft. 6-in.; therefore, it seems far more likely that, if the change could be brought about, Brazil would prefer to change to 4 ft. 8½-in. as a standard, thus conforming with Uruguay and Paraguay and the existing lines in Entre Rios and Corrientes.

For the 19 000 km. of narrow-gauge lines now existing in Brazil it would cost about $40 000 000 or $50 000 000 less to convert them to standard-gauge than to 5 ft. 6 in. Looking toward the north, also, the lines of Peru are largely standard-gauge, even where they cross the highest passes of the Andes, and this really is the only logical gauge to adopt if an agreement could be effected between the different countries.

The conversion of the meter-gauge lines of the Argentine to standard 4 ft. 8½ in., and the adoption of that gauge for all new lines north of Santa Fé and Cordoba, and in the rest of the country, except for the legitimate extensions of the existing broad-gauge lines, would be a much less costly proceeding than that estimated above for their conversion to 5 ft. 6 in. It would provide an efficient transportation machine reaching from Buenos Aires to all parts of the country, and in harmony with the network already existing in Uruguay, Entre Rios, Corrientes, and Paraguay, and it might not be unreasonable to expect that, with this in view, Brazil would join in the movement, as there can be no question that such large areas as those under consideration cannot be efficiently developed by a transportation machine inferior to that which has achieved such wonderfully successful economic results in the United States, with which country this part of the world is most nearly comparable.

Making the estimate for the Argentine alone, on the basis of a change to 4 ft. 8½ in. instead of 5 ft. 6 in., and leaving out the existing 5 ft. 6-in. system, the cost of the completed system of standard-gauge roads would be as shown in Table 24.

TABLE 24.—Cost, in Gold Dollars.

Conversion of 9 000 km. of meter-gauge to 4 ft. 8½-in. at, say, $8 500 per km.	$76 500 000
Additional cost of building, say, 9 000 km. of 4 ft. 8½-in. instead of meter, at $2 000 per km.	18 000 000
Extra cost of building 2 500 km. of extensions in Entre Rios, Corrientes and Missiones	nil.
Cost of changing existing 2 500 km. in Entre Rios, Corrientes and Buenos Aires	nil.
Total	$94 500 000

That is, the standard-gauge system in existence at the end of 20 years would have cost about $4 000 per km. more than if the meter-gauge had been perpetuated. The interest on this at 6% would be about $240 per km. per annum, which it is believed could easily be saved by economy of operation, to say nothing of the necessity of having an adequate transportation machine.

Next, take the area south of 22° South Latitude and between the Atlantic Coast and a line drawn from Buenos Aires along the river to Santa Fé and passing through Tucuman and Salta to Sucre, but omitting the section north of São Paulo and Rio, already fairly well served by railways. There are in this area about 1 000 000 sq. miles, served by only about 12 200 km. of railroad or 1.22 km. for every 100 sq. miles of territory.

Of this area, the Province of Entre Rios is, perhaps, the best served at present, with about 3.9 km. of railway per 100 sq. miles, and it is seriously proposed to double this mileage within the next few years.

The railway mileage of the Argentine has increased three-fold in the 20-year period, 1890 to 1910, and it seems not unreasonable to expect the same rate of increase during the next 20 years in the area under consideration. It is true that the climatic conditions are not quite so favorable, but, on the other hand, the era of great expansion in North America, through the development of the country by new lines of railway, is now over, the world demand for pastoral and agricultural products is increasing, even the United States itself is just entering into the class of consumers, so that the development of this area seems inevitable, and will only be held back, if at all, by the lack of capital.

Assuming that there will be 25 000 km. built in this territory during the next 20 years, which will give a density of 3.7 km. per 100 sq. miles,

of this, probably, 5 000 km. will be of standard-gauge, in any event. Then the cost of a standard-gauge system throughout the whole area, including the cost of conversion of all the existing lines except those of broad-gauge of the Argentine, would be as shown in Table 25.

TABLE 25.

Cost of conversion of 28 000 km. of meter-gauge to standard, at $9 000	$252 000 000
Additional cost of building 20 000 km. of line, 4 ft. 8½-in. instead of meter, at, say, $2 500	50 000 000
Cost of conversion of 500 km. of 5 ft. 3-in. to 4 ft. 8½-in., at $1 000	500 000
4 800 km. in Uruguay, Entre Rios, Corrientes, and Paraguay not requiring change	nil.
5 000 km. of new line in Uruguay, etc., etc.	nil.
Total additional cost	$302 500 000

It will be noted that somewhat larger allowances for conversion, etc., are made for this whole system than for the Argentine alone.

The standard-gauge system in existence over this whole area at the end of 20 years, therefore, would cost about $5 000 per km. more than if the meter-gauge were perpetuated, this involving an annual interest charge of, say, $300 per km. The greater cost by including the lines of Brazil is due to the much greater present development of the meter-gauge system there, and also by additional allowances to cover the additional costs, as a great deal of the topography is much more accidented in Brazil than in the Argentine. On the other hand, no account has been taken of the vast areas in Brazil north of Latitude 20°, in much of which, especially in Eastern Brazil, there is likely to be considerable development by new lines, which, if built in the first place as standard-gauge, would reduce the average cost per kilometer of the whole system.

It seems to be not unreasonable to assume that, as many of the narrow-gauge lines will require stone ballast at a cost of from $1 500 to $2 500 per km., where the wider gauge could be operated just as well without, the completed system should be given some credit on this account, which might easily be as much as $1 000 per km. over the whole system, thus reducing the interest charge by $60 per km.

There is also the question of double-tracking many of the narrow-gauge lines, which would not be necessary if they were medium- or broad-gauge, and this would offset quite a little of the cost of conversion, though it is somewhat difficult to say just how much. Suppose, how-

ever, that 15% of the systems as narrow-gauge would have to be double-tracked and half of this, say 4 500 km., could be avoided by the change of gauge, there would be a saving of about $100 000 000 on this item alone, or one-third of the estimated additional cost of the whole system as standard-gauge.

It has been shown that, on the basis of the most conservative estimates, the saving in cost of operation made possible in only the one item of larger rolling stock, might easily be more than $300 per km. per annum, and as it is considered necessary even now to use 70- and 80-lb. rails on the narrow-gauge, this is not offset by the necessity of increasing the strength of the track to take care of the heavier rolling stock, as this weight of rail is ample to carry the ordinary standard-gauge equipment of fairly heavy types, on which these estimates are based.

It seems almost superfluous to point out the advantages of unification of the gauge. Besides the inconvenience of transfer and the actual cost of moving freight from one car to another, which has been variously estimated at from 5 to 10 cents per ton in India,* there is the actual damage to many classes of freight, such as fruit, etc. The transhipment of refrigerated products is impractical. Two cars are standing idle while any transhipment is being made, there are the unavoidable delays in yards waiting for the necessary cars of both gauges to be provided and shifted into position, and, above all, the loss in fluidity of rolling stock, which would be especially noticeable in both the Argentine and Brazil, where, for a few months during and after the harvest, there is an exceptional demand for rolling stock which could be used during the other seasons for hauling timber.

In the Argentine this question of transfer from one gauge to another has not become acute up to the present time, as most of the lines of each of the gauges reach terminal points on the water-front, to and from which most of their traffic flows. With the development of the country, the actual interlocking of the lines of the different gauges, and the present tendency to continue this mixing of the territories, this question, however, is bound to assume greater and greater importance, and the necessary transfers will be the cause of more and more inconvenience and delay.

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* *Minutes of Proceedings*, Inst. C. E., "The Railway Gauges of India," by Sir Frederick Robert Upcott, Vol. CLXIV, p. 196.

It is true that the foregoing figures are based largely on assumptions, and may be varied widely in many individual cases, but it is believed by the writer that on the whole they represent a fair average, and at least tend to indicate that the adoption of 4 ft. 8½ in. as a standard, and the conversion of the existing narrow-gauge lines, can be justified on economic grounds and approximately so from the financial viewpoint.

It is hardly to be expected that these figures are definite enough to justify private capital in entering on an enterprise of such magnitude, but, taking into consideration the fact, which it is believed has been demonstrated, that the narrow-gauge is absolutely inadequate for the development of such large areas as those under consideration in the Argentine, Eastern Bolivia, Brazil, Paraguay, and Uruguay, to say nothing of the rest of South and Central America, it may not be unreasonable to expect such a measure of assistance from the Governments involved, which would be justified by the benefits they would derive from a system of railways of uniform gauge covering the whole area.

This argument, it will be seen, is based largely on the supposition of the continued and continuous development of these countries, which seems assured, provided the past policy of encouragement and appreciation of private capital is continued and the political situation remains as stable as it now seems to be. Without foreign capital, of course, this development will be tremendously retarded, to the benefit of competing countries like South Africa, Australia, and New Zealand.

It has been, and still is, the policy of the Governments of both Brazil and Argentine to build as State enterprises lines of railway to connect up or push the development of certain sections not sufficiently attractive at the time in themselves to private enterprise. Of course, the extent to which this can be carried out is more or less limited, as in all these newer countries there is a demand for more capital than is available; therefore, the Government lines, after a while, are sold or leased, and the capital thus acquired is used for further development elsewhere. It seems to the writer that, taking everything into consideration, such lines, if built to standard-gauge, would cost little if any more than have the meter-gauge lines, and there is no question at all but that they would be a far more valuable asset because of their undoubted greater economy in operation and their better salability

because of the wider market for any standard article than for one of inferior quality.

The 5 ft. 6-in. lines of the Argentine probably will not consider the change for the present, but, as they are fairly well confined to a limited area, it will do no harm to leave them as they are. They are efficient, and to attempt to include them in any scheme for unification of gauge would be to create a formidable opposition which it would be difficult to overcome. The adoption of 4 ft. 8½ in. as a standard for new lines, and the gradual change of the meter- and other narrow-gauge lines to the standard, however, should be established as the policy, if the requirements of the future are to be met.

It must be borne in mind that, as the distance from the seaboard increases, the necessity for cheap transportation becomes more and more important, and as this section of the world must depend for its future prosperity on its ability to compete successfully in the food markets of the world, cheap efficient transportation is a *sine qua non*. The United States has achieved results in this line which are far ahead of those obtained elsewhere, almost wholly by reason of the increase in train loading. It is quite certain that no narrow-gauge line can attain anywhere near the economic results obtained in the United States, and nothing which falls much short of this will solve the problem for the area under discussion, which only needs capital and adequate railroad transportation to show the most phenomenal development in the history of the world. This development, however, will be tremendously handicapped with anything less than the efficient, economical transport which can be furnished by standard-gauge railways.

In closing, the writer wishes to express his thanks to Mr. Percival Farquhar, President of the Argentine Railway Company, for permission to use much of the information contained in a report made to that company on this subject by the writer, to Mr. L. E. Young, of the Middletown Car Company, for data in regard to rolling stock, and to Mr. C. M. Muchnic, of the American Locomotive Company, for data in regard to locomotives and other information. It is believed that proper credit has been given in the text for quotations and assistance from engineering periodicals and other sources, but inasmuch as a fairly wide search of the literature of the subject has been made,

it is possible that some omissions of credit may have been made, if so, it is needless to say it has been unintentional.

Generally speaking, it is believed that the statistics given in the paper are substantially correct, and in any event such differences as there are do not affect the general argument, so far as the gauge question is concerned. As with all figures relating to railroads, there are often differences due to different methods of calculation or recording, and this is perhaps especially so in regard to South America. The writer, therefore, would esteem it a favor if those who may have more intimate knowledge of any facts will not hesitate to state them with a view to making the record as complete and accurate to date as may be.

APPENDIX A.

History, Experiences of Other Countries, Etc.

The following notes give briefly some account of the unification of the gauge of the railways in the United States and Mexico, the experience with lines of different gauges in India, Australia, etc., and Table 26 shows the length of lines of various gauges now in operation in the different countries of the world:*

TABLE 26.—Length of Railways of Various Gauges in Operation Throughout the World.

All Distances in Kilometers.

	Broad, more than 5 ft.	Medium, 4 ft. to 4 ft. 11 in.	Narrow, 3 ft. to 3 ft. 11 in.	Industrial, less than 3 ft.
Great Britain	4 959	40 017	909	104
Europe (Continent)	71 142	206 912	23 517	8 264
North America	.....	447 721	12 394	.....
Central America and West Indies	153	3 382	2 596	141
South America	18 852	6 663	37 115	3 094
Asia	43 781	11 650	41 299	2 618
Australasia	6 415	6 179	19 257	226
Africa	.....	5 275	27 843	4 039
Totals	145 302	727 799	164 930	18 486

Note.—The medium-gauge is nearly all 4 ft. 8½ in.
The narrow-gauge is nearly all meter or 3 ft. 6 in.

United States.—The following, prepared by Mr. George L. Fowler, of Messrs. Hildreth and Company, gives a brief statement of the history of the gauge discussion and its result in the United States.

"The narrow-gauge mania which possessed the railroad builders of the United States in the early Seventies was born of the bargain hunting desire to get something far below its real value. It was at the time very difficult if not impossible to raise money for the construction of new railroads of the standard-gauge. It was just after the civil war, and hard times were upon the country, yet in spite of this fact, the people over the whole land were clamoring for a more rapid development of the resources than was taking place, and this clamor was especially loud in its demand for an increase of railroad construction, but capital could not be obtained.

"It then occurred, to those who were most vitally interested in these constructions, that if cheaper roads could be built, the money for their construction might be obtained. They immediately, therefore, jumped to the conclusion that, because a gauge of 36 in. is only about three-fifths of 56 in., a road of that gauge could be built for a correspondingly lower price, and straightaway positive statements were made to that

* Compiled from "Universal Directory of Railway Officials," 1912.

effect, which were honestly accepted by a large number of men. It was assumed that, because the gauge was to be narrowed, all costs would be cut down in a corresponding degree. It was argued that because of the narrow-gauge, the cars would be made lighter than those which it had been found advisable to use on the broad-gauge, forgetful of the fact that equipment of the same, or even lesser, weight could and had been built and used on the standard-gauge lines, and had been found unsatisfactory.

"The whole case rested, apparently, so far as the United States was concerned, on the inability to raise money for ordinary railroad construction, and the narrow-gauge seemed to promise greater dividends on a given investment. It was characteristic of the whole discussion that the advocates of the narrow-gauge insisted throughout that their construction was the cheaper, and could not or would not see their opponents' side of the case, which was, that the narrow-gauge could not be built materially, if any, cheaper if the same facilities and capacity for the transportation of freight and passengers were to be afforded. They failed to see or acknowledge that a light locomotive and car could be built for a broad-gauge line.

"The physical disadvantages of the narrow-gauge, such as cramped storage and seating capacity, the greater instability of the rolling stock, and, above all, the chief drawback of the impossibility of interchanging cars with the standard lines, with the consequent extra cost and delays in transportation, were pointed out again and again, but in spite of all this the narrow-gauge construction went on for several years, and, until the money market eased, made a great headway.

"Meanwhile, the disadvantages of different gauges were making themselves manifest, and it was also being realized, with great rapidity, that heavy loads meant lesser costs than when light ones were hauled, so that between 1873 and 1885 there was a very rapid increase in the carrying capacity of the standard-gauge cars, rising as it did from 10 to 30 tons. With this rise the dead weight of freight cars dropped from a ratio of 1 lb. of dead weight to 1 lb. of paying load to ½ lb. of dead weight to 1 lb. of paying load.

"When the necessity of car interchange drove the broad-gauge lines to change to the standard (4 ft. 8½-in.) gauge, the narrow-gauge lines were left in a state of greater isolation than they had been at the outset of the movement.

"Finally, the recuperation after the panic of the Seventies made money more easily procurable for railroad construction, and the narrow-gauge roads, which had been built and equipped in the lightest and cheapest possible manner, found themselves doubly handicapped by their lack of facilities to do business, and by their isolation.

"The result of all of it was that, the furore past and the money market relieved, the narrow-gauge mania died a natural death, if a death from lack of substance can be called natural. The result then, of the forty years of practical experience that has elapsed since the battle of the gauges was precipitated in the early Seventies, has been to quite discredit the narrow-gauge system. It has been proven beyond all peradventure that the narrow-gauge railway cannot be operated so as to keep its ton-mile costs down to the figure obtained on the standard-gauge roads, and this has been the cause of its demise.

"Cars and locomotives of present capacities are impossible on a 3-ft. or 3 ft. 6-in. gauge, and time has shown that the contentions of the early opponents of the system were correct and that the narrow-gauge railroad cannot compete with the broad-gauge in capacity, facilities offered, or in cost of operation, when this cost is put upon the ton-mile basis."

The Denver and Rio Grande was the most important of the narrow-gauge lines changed to standard, and is interesting from the fact that it is a mountain line, of steep grades, very sharp curvature, and heavy work, crossing as it does the mountainous region of Colorado from Denver to Salt Lake City.

The change was started in 1886, during which year almost half the mileage (1 200 miles) was changed, the rest being done as the finances of the road permitted, and less than 6 months ago (September, 1912) it was decided to complete the last section, over Marshall Pass, a distance of 236 miles, the cost of which work, with the improvements in the line, is estimated to be about $2 000 000, or about $8 500 per mile.

It is of interest to note that on this line, which is now a link in a transcontinental route, and reaches an elevation of more than 10 000 ft., there are grades of 3.8% and curves of 150 ft. radius operated by heavy locomotives, and on which passenger trains of standard Pullman equipment are operated.

Many other lines in the United States were changed, but the largest mileage was in changing from 4 ft. 10-in., 4 ft. 11-in, and 5 ft. 0-in. to the standard 4 ft. 8½-in., this involving some 25 000 miles of various lines, mostly in the South. The total mileage of narrow-gauge changed to standard was probably about 5 000.

Mexico.—The principal railways in Mexico were built to standard gauge, 4 ft. 8½ in.; the gauge of the Mexican National, however—the original trunk line from the frontier of the United States to the City of Mexico, 1 200 miles in length—was made 3 ft. 0 in. Notwithstanding that it had a much shorter direct line to the United States, it was unable to compete with the Mexican Central and the International. In his Annual Report in 1889 the President of the Company said:

"Having been brought by experience to a realization of the inadequacy of a narrow-gauge road to develop a thoroughly satisfactory transportation service for a large volume of business * * * induced your management to take up some time ago the study of the practicability and desirability of changing the track to standard-gauge."

The line was changed to standard gauge between 1902 and 1904.

India.—The gauge of the Indian Railways has been the subject of almost endless discussion. Although nearly all who are familiar with the subject agree that a difference in the gauge of continental railways, or in countries as large as India, is unfortunate, all the discussion and the various reports of eminent engineers and transporta-

tion experts have led nowhere, and lines of both gauges are continuing to be built, apparently, however, only because the problem of conversion seems to be too big to handle. A few quotations from some of the discussions will show the attitude of some of the most prominent British engineers who have studied the subject. The matter was brought up officially for discussion by the Institution of Civil Engineers in 1906, the paper and discussion at that time occupying 135 pages of the *Minutes of Proceedings*, and, previous to that, at intervals as far back as 1873,* the following, at that time far-sighted, opinion was expressed:

"While the 3½-ft. gauge might answer for the carriage of heavy minerals in special districts, the general commerce of every populous country mainly consists of articles of low or medium specific gravity, such as food, clothing, fuel, etc., averaging about 80 cu. ft. per ton weight, for which the 5½-ft. gauge is in every respect most suitable as regards costs, stowage, safety, economy, the intricate elements of military defense, and the power of adopting single-track lines of railways for the accommodation of a large amount of traffic."

Mr. Thomas Robertson was appointed in 1903 as Special Commissioner by the British Government to report on the administration and working of Indian Railways, and in his report he pointed out the difficulties of working the two gauges, meter and broad, as follows:

"It will generally be admitted that a break of gauge is a drawback always, and that in certain eventualities it might prove extremely inconvenient. It necessitates a great deal of expense in the transshipment of traffic and a very much larger supply of rolling stock, since the unavoidable delays during the progress of transshipment absorb a large quantity of rolling stock of both gauges.

"The question is so full of difficulty and the evil is so far advanced that it is not easy to advise on it.

"Uniformity of gauge is bound to be demanded some day, and the longer a settlement of the policy to be pursued is deferred the greater will be the difficulty and expense of introducing it."

He discusses the relative merits of the two gauges, and favors the meter-gauge, largely because of the greater width of rolling stock in proportion to the width of the gauge. (This additional width, as is shown elsewhere, is not an unmixed blessing, as it is gained at the expense of safety and stability.) He concludes by advocating the unification of the gauge by the adoption of the medium-gauge, 4 ft. 8½-in.

In 1906 a paper was presented before the Institution of Civil Engineers of Great Britain by Sir Frederick Robert Upcott, Chairman of the Indian Railway Board, who pointed out the facts in regard to the gauge of Indian Railways without much comment. They were stated to be:

* And in *The Railroad Gazette*, November, 1872.

That the internal communications of the country were more important than any considerations of communication with other countries;

That the bulk of imports and exports was by sea;

Large internal trade;

The attempt to confine the narrow-gauge to a particular zone was useless, as these lines must find an outlet to the coast, and this created a great confusion at the ports by reason of difference in gauge;

The trouble and inconvenience of transshipment were considerable, and would by no means be measured by the cost of actual transfer of articles, but involved rolling stock standing idle, lack of fluidity of rolling stock, etc., etc.

Some of the discussions were, briefly, as follows:

Sir Guilford Molesworth had nothing to say against the meter-gauge as a gauge, but he did think the standard-gauge (5 ft. 6-in.) was better suited to the bulky agricultural traffic of India.

Sir George Bruce stated that when the discussion of this question came up in 1873 he estimated that the difference in cost of construction was very large, being based on the misconception that the width of gauge was the element which regulated the cost. It did nothing of the kind. With a coach of a given width, nothing was gained by putting the wheels closer together.

Mr. R. W. Egerton, commenting on a suggestion to change broad-gauge to narrow-, said:

"It would be economically unsound to substitute an inferior means of transportation for a superior one, at the same time increasing the capital cost * * *. The meter gauge, on account of the low limit of speed attainable, was an unsuitable gauge for large trunk-lines."

Mr. F. E. Robertson observed that most discussions on the gauge question had been vitiated by the implicit assumption that all the dimensions were geometrically similar, and, also, by the omission to remember that speed was a commodity which was sometimes worth money.

The difference in running speeds between the two gauges was not really known, for the broad-gauge (in India) was in this respect usually worked in such a leisurely way, that the meter-gauge had no difficulty in keeping fairly close to it.

Sir Frederick Robert Upcott, replying to a statement that high speed was not required in India, stated:

"Mr. * * * was incorrect in his views as to the need of rapid travelling in India. India, * * * was like every other country, and had progressive ideas of speed, comfort and safety, and the Railway Administrations would have to meet these needs for all classes."

Africa.—In connection with this same discussion of the gauge of the Indian Railways, there were the following expressions of opinion in regard to the gauge of the African Railways:

The General Manager of the Cape Railways wrote (1906):

"There has been considerable agitation on the subject of the laying of light railways during the last few years. I see no reason for changing the opinions I expressed in 1891 and 1892 on this subject, and I trust that Parliament will gravely consider the probable effect of the introduction of a narrower gauge than that of our own railways before authorizing such a breach of gauge. If the gauge originally adopted, 4′-8½″, had been continued instead of changing to 3′-6″ the journey from Capetown to Johannesburg would be performed in about half the present time. The effect of such saving of time in the passenger traffic would have been enormous, but if the present gauge is to be still further reduced, what will posterity think of the foresight, or want of foresight, in adopting a standard that limits the speed and carrying capacity of the trains? When the traffic is not expected to be heavy I see no reason why railways should not be constructed with less ballast than a standard line. When the traffic improves, the line could be better ballasted and prepared for quicker speeds and heavier loads."

C. O. Burge. "The line from the Zambesi river through to Cairo had been projected on a *miserable 3′-6″ gauge.* It would be found that the steamship companies, who were progressing faster than the railway projectors, would carry passengers from North to South Africa much more quickly by steamer than the railway could on a line of 3′-6″."

Japan.—Japan, after a most careful examination of the question, has decided to change the gauge of its railways to 4 ft. 8½ in. The 3 ft. 6-in. lines of that country were found to be entirely inadequate for the transport of troops and war material during the Russo-Japanese war, and, after a most careful consideration of the whole question, it has now been decided to make the change, the estimated cost of which is said to be about $150 000 000 for the 5 000 miles of line.

The South Manchurian Railway was originally 5 ft. 0-in. gauge and during the Russo-Japanese war was converted by the Japanese to 3 ft. 6 in., this being chosen because only rolling stock of that gauge was available. After operating it as a narrow-gauge road for 2 years, however, it was changed to 4 ft. 8½ in., corresponding with the gauge of the Korean Railways, and it has been stated that the Japanese have felt that they owed a debt of gratitude to the American engineers who built the first Korean railway and thus established a gauge which was of great service to them during the war, the 3 ft. 6-in. gauge which they were obliged to adopt temporarily for the South Manchurian Railway having been found entirely inadequate.

Australia.—The situation in Australia is described in the following quotation:*

* *Engineering Record*, September 28th, 1912.

"By some unfortunate misunderstanding, each now laying the blame on the other, New South Wales and Victoria started railways in the Fifties on different systems, the first on the standard, or 4 ft. 8½-in., and the latter on the 5 ft. 3-in. gauge, and, notwithstanding the advice of the engineers of the day, persisted, until now we have two great groups of over 3 500 miles each * * *. The slight difference of 6 in. stands much in the way of even the very inadequate remedy of a mixed gauge, owing to the difficulty and expenses connected with the construction and working of the switches and frogs in such a case.

"Subsequently, Queensland and Western Australia, attracted by the expected but exaggerated economies in the construction and working of narrow-gauge lines, which were too hastily assumed to be synonymous with light lines, adopted 3 ft. 6 in., and South Australia, after most of its main lines had been made to correspond with the width [5 ft. 6 in.] of those of its neighbor Victoria, introduced a local diversity by making its extensions and branches on the smaller, 3 ft. 6-in., basis.

"Unlike the United States and Canada who repented comparatively early in this matter, but at a cost of a large amount of money, Australia has done nothing but talk about it, and, owing to the enormous extension of each gauge in recent years, it is quite out of the question now to think of universal conversion, which might cost anything up to $60 000 000."

Finally, after the federation, it was decided to build a transcontinental line to unite the extreme easterly and westerly States of the Commonwealth, and, of course, the question of the most suitable gauge came to the front at once. After long debate and after calling to its aid the most expert advice, both engineering and other, that it was possible to obtain, the 4 ft. 8½ in. was adopted.

There still remained in abeyance the question of the gauges of the existing lines, but it has just recently been announced* that it has been decided to unify the gauges of all the lines, adopting 4 ft. 8½ in. as a standard and, in view of the general benefit of this to the whole country, the greater part of the burden of this change, estimated to cost about $175 000 000, will be borne by the Commonwealth as a whole.

As an example of the misleading statements often made in discussing the relative value of the gauge, the following is worthy of notice because it is so recent, and made by what one would suppose might be a competent authority.

In December, 1912, speaking at the Royal Colonial Institute in London, Sir Thomas Robinson, Agent General of Queensland, in describing the 3 ft. 6-in. railways of that part of Australia, made the statement that "Queensland had constructed 4 266 miles of railways of this gauge at practically half the cost of 3 807 miles of 4 ft. 8½-in. gauge in New South Wales." He omitted, however, to mention the fact that the total net earnings of the latter were more than double

* *Railway Age Gazette*, July 11th, 1913.

those of the former. He also stated that "in the working of the railway the comparatively light weight of the rolling stock gave it a great economic advantage over railways constructed on the wider gauge," yet said later, "Coming to rolling stock, the lighter engines and cars of the early days ran on lighter sections of rails, but the increased weight and power of the engines had necessitated the relaying of the permanent way with far heavier rails, * * * and the use of this heavier rail was steadily and surely extending from one end of the State to the other." He claimed that break of gauge has few disadvantages, yet stated that a new route has been laid out to the border of New South Wales on which "all the stations, tunnels, etc., had been built sufficiently wide so as to run the New South Wales trains (4 ft. 8½ in.) right through".

The consensus of all these opinions, from a wide range of territory and a very large number of prominent men, covering practically the whole of the railway systems in the English-speaking parts of the world, is decidedly against the adoption of lines of different gauges in the same country, and almost entirely in favor of the broad (4 ft. 8½ in. or 5 ft. 6 in.) as compared with the narrow (3 ft. to 3 ft. 6 in.).

APPENDIX B.

GENERAL DESCRIPTION OF THE TOPOGRAPHY, PHYSICAL CHARACTERISTICS AND BUSINESS CONDITIONS OF SOUTHERN SOUTH AMERICA; THE DEVELOPMENT AND PRESENT SITUATION OF ITS RAILWAYS.

In the following, the Argentine is referred to perhaps more particularly than any of the other countries, but this only because the writer is more intimately familiar with it, and the data he has available relate principally to the railways of that country. The conditions there, however, are quite generally similar to those elsewhere in this south temperate zone, and may be assumed to be applicable throughout most of this area and to its railways.

The northern limit of this section for convenience may be assumed to be about Lat. 20° S., that is to say, approximately equal to that of the southern coast of Cuba and the City of Mexico; its southern limit will be taken as the City of Bahia Blanca, Lat 39° S., which is equivalent to that of Baltimore. The latitudes of some of the more important cities may be compared with some of those of the northern hemisphere, as follows:

Rio de Janeiro,	Havana,
Santos and São Paulo,	Key West.
Antofogasta,	Key West.
Asuncion,	Cape Sable, Florida.
Valparaiso and Santiago, Chile,	Atlanta, Ga.
Buenos Aires and Montevideo,	Memphis, Los Angeles.
Bahia Blanca,	Baltimore, Denver.

It should not be inferred from this that there is no development in that part of the Argentine south of Bahia Blanca and Neuquen, and including Patogonia.

The railway development in this section, however, is not large, and is most likely to be dominated, as far as gauge is concerned, by that of the four important lines in the Province of Buenos Aires, which are 5 ft. 6 in.

The principal topographical features are the main range of the Andes and the River Parana, with its tributary the Paraguay, and the River Uruguay. The Andes occupy a comparatively narrow strip along the Pacific Coast, some 150 to 200 miles wide, broadening out at the northern end of Chile and in Bolivia. Between this range and the Parana and Paraguay Rivers is the vast almost level plain reaching from considerably south of Bahia Blanca to well up into Bolivia, which comprises nearly the whole of the Argentine. Uruguay, Paraguay, the two western provinces of the Argentine, Entre Rios and Corrientes, and the territory of Missiones, are considerably rolling to hilly. In Brazil the land rises abruptly from the coast line to elevations of 2 000

ft. and more, and then slopes generally toward the Rivers Uruguay and Paraguay, the intervening country being mostly rolling and in some parts rather broken and accidented.

In the Argentine there are few, if any, rivers of importance between the Parana and the Andes, the drainage from the easterly slopes of the latter, which is considerable, disappearing in the sandy soil before it has traversed the plain for any considerable distance. Toward the north, above the confluence of the Paraguay and Alto Parana, there are two important tributaries, the Bermejo and the Pilcomayo, the latter forming the northern boundary of the Argentine and dividing it from Paraguay and Bolivia. East of the Parana and Paraguay the country is generally well watered and the bridging of the rivers is an important item.

Lest the foregoing should convey the idea that much of the area of the Argentine is arid, it may be well to note further the peculiar geographical formation of this plain. The top covering of soil, which is of the richest character, and similar in appearance to that of the so-called "black waxey" belt of Northern Texas, at the Parana River and for some distance back from it, changes to a more friable and at times almost sandy loam farther back, and is underlaid by the so-called Pampean mud or "Tosca", a hard, indurated clay, almost shale. The water from the mountain finds its way between the top soil layer and the "Tosca" and between the successive layers of this latter, and is found almost everywhere by comparatively shallow borings. Nearly all of this is good or at least fair drinking water, but some is alkaline or slightly saline, and, therefore, bad for locomotive uses.

The railroad development of the Argentine, therefore, has gone ahead under the most favorable conditions, as far as facility of construction is concerned, and there has been practically no excuse at all for the adoption of the narrow-gauge. Some of the Government lines, notably that running from Tucuman north to La Quiaca, on the border of Bolivia, are in a mountainous country, but the proportion of the total is very small, and may be imagined when it is stated that more than 90% of the alignment is tangent.

Perhaps the most important physical characteristics which affect railway construction and operation are the scarcity of stone or gravel and the high cost of fuel. The coal that is used has to be imported from Europe. The scarcity of stone makes the question of ballast important, as the broad- and medium-gauge lines stand up much better with only earth ballast than do the narrow-gauge lines, and in comparing the quality of the track, the experience of the writer with the railways of the Argentine, leads him to think that it is almost fair to say that a fair stone-ballasted narrow-gauge track is no better than a good earth-ballasted wide-gauge.

The Argentine, as is perhaps well known, derives its wealth from

its pastoral and agricultural products. There are no manufactures of importance, as there is an almost entire lack of minerals and fuel, except wood. That part of the Argentine which, thus far, is developed most, as may be seen by the density of the railway lines, is in the Province of Buenos Aires and the southern parts of the Provinces of Cordoba and Santa Fé. This is an almost treeless plain, formerly devoted to little else than cattle raising, but now gradually being converted to agricultural uses, and the raising of the finer grades of cattle and horses, alfalfa replacing to a large extent the original native grasses. Wheat is cultivated mostly in the southern part of the Province of Buenos Aires and in the vicinity of Bahia Blanca, and exported largely from that port. Maize and linseed are the important crops farther north and in Entre Rios. The principal refrigerating and packing plants are on the Rio de la Plata and the River Parana between La Plata and Rosario. Quebracho comes from north of the City of Santa Fé, and nearly all the exports of it are from that port.

With the great valorization of the lands in this section, due to its agricultural development, the cattle industry is being pushed toward the north, and there seems to be little doubt that the world will soon have to look to this section for much of its meat supply, for the production of which there are vast areas of grass lands available in that part of the Argentine north of Santa Fé and reaching up into Bolivia and Paraguay, in many parts of this latter country, in Uruguay, and in Southern Brazil.

Up to the present time, nearly all the structural timber used in the Argentine has been brought from Europe and the United States, but it is now known that there are large forests of pine and cedar in the States of Parana and Sta. Catharina, in Brazil and in the Territory of Missiones. In Paraguay there are forests of important hardwoods which will now be made commercially available, as there has just been opened up through railway communication between Asuncion, through Corrientes and Entre Rios, to Buenos Aires, and a new line is under construction from Villa Rica on the Paraguay Central toward the east which will probably open up important timber industries and reach the forests where much of the yerba maté grows.

At Tucuman, and farther north between Salta and Embarcacion, there are important sugar industries, and the business of raising early fruits and vegetables for the markets of Buenos Aires, Rosario, Montevideo, etc., is already assuming considerable proportions. Mendoza and San Juan are the centers of quite important grape and wine industries, producing annually some 3 000 000 hectoliters (66 000 000 gal.) of wine, in addition to the grapes which are shipped in very large quantities during the season all over the country and even to Europe and the United States.

The great and important product of the so-called Gran Chaco, which

embraces in the Argentine practically all that territory in the triangle with Santa Fé as its apex and the Pilcomayo River as its base, is quebracho. These trees are not unlike scrub oak in appearance and are generally too twisted and crooked to be of use for structural timber. The wood is very hard and durable, and is used for railroad ties, fence posts, and telegraph poles, the trimmings, roots, etc., for firewood, and the smaller branches for charcoal. It is very rich in tannin, and the whole logs and the extract are shipped in large quantities to Germany for tanning purposes. The whole area of the triangle referred to, so far as known—and quite a little of it has not yet been explored—comprises large areas of forests with much quebracho interspersing the open spaces where the rich grasses provide excellent grazing for cattle. There are some parts which are alkaline, some swampy, some waterless, but, on the whole, it is believed that the greater part of it is useful especially as a cattle country and where maize, for feeding on the spot if not for export, linseed, cotton, tobacco, sugar cane, etc., can be grown. The quebracho forms the bulk of the freight for the Santa Fé Railway and the southerly end of the Government lines, and this is a product which demands rolling stock of large capacity for its transportation. It is of considerable importance, as it provides paying freight for new lines from the beginning, while the country is being developed pastorally and agriculturally.

Buenos Aires is by far the most important city in the Argentine. It is both the National Capital and the commercial center, its population, 1 300 000, is one-fifth of the total of the country; it receives 80% of the imports, and the exports passing through it are 60% of the total.

The principal other ports are Rosario, the second most important city, with a population of about 100 000, La Plata, the capital of the Province of Buenos Aires, Bahia Blanca, and Santa Fé. The imports which come to these places are mostly coal and lumber, but cereals are exported from all of them as well as from quite a number of smaller places along the rivers where ocean-going steamers can lie alongside the bank and load, being served by short spurs of the railroads tapping the districts adjoining. The Port of Buenos Aires is served by a Port Railway of 5 ft. 6-in. gauge operated by the Government, with a strong feeling against admitting any other gauge. The Port of Bahia Blanca has been developed by the railroads, is served by three broad-gauge railways over their own lines, and has had a very rapid development. The Ports at La Plata, Rosario, and Santa Fé, have mixed gauge (5 ft. 6 in. and meter), the first and last operated by the Government, and Rosario by a French Corporation under a concession for its exploitation.

Most of the merchandise is imported through Buenos Aires and distributed from that city throughout the country, the imports passing through this port being about 6 500 000 tons for 1910, and the exports about 2 000 000 tons. A small proportion of the imports, about

25% of the total tonnage, and consisting principally of lumber and coal, comes directly to Rosario, and in much lesser quantities to Bahia Blanca, La Plata, and Santa Fé, with a scattering to some of the smaller ports.

The cereals are exported principally from Rosario and Buenos Aires in about equal quantities, with Bahia Blanca the next in importance, and then a scattering from a considerable number of smaller ports along the Parana and a few on the Uruguay. The cereals flow naturally to the nearest seaport, ocean-going steamers reaching as far as the City of Santa Fé on the River Parana and Concepcion del Uruguay on the Uruguay River.

Cattle (live) are shipped from all over the country (see Fig. 5) for distribution to the packing plants on the river between La Plata and Rosario. The cattle from Corrientes and parts of Entre Rios go to the plants along the Uruguay River which take, principally, the inferior grades for extracts, dried beef, etc. There is also a considerable movement of cattle from one section to another for growing, fattening, etc., and for shipment to the west coast by driving across the mountains in summer.

There is good passenger business on nearly all lines, sleeping and dining cars being in general use.

The conspicuous feature of the freight traffic, of course, is the cereals. There is a heavy demand for cars during the shipping season, February to June, and in this period quite a large proportion of cars are returned empty, this unbalancing of the traffic being noticeable on nearly all the lines.

In Brazil, of course, the most important products are coffee and rubber, the former coming from that section of the country to the north of São Paulo and Rio. This territory, as will be seen by Plate I. is served by a fairly dense network of railways which, however, are of two different gauges.

The following table shows the quantities of the principal items of export for 1906. The cotton, sugar, cocoa, and tobacco, are mostly from Pernambuco and Bahia to the northeast of Rio; rubber, of course, comes from the Amazon, and the other products from the States to the south of Rio. The quantities are long tons.

Sugar	85 000	tons.
Cotton and cotton seed	35 000	"
Rubber	62 200	"
Cocoa	25 000	"
Tobacco	23 600	"
Coffee	840 000	"
Maté	57 800	"
Flour and bran	31 500	"
Hides	27 500	"

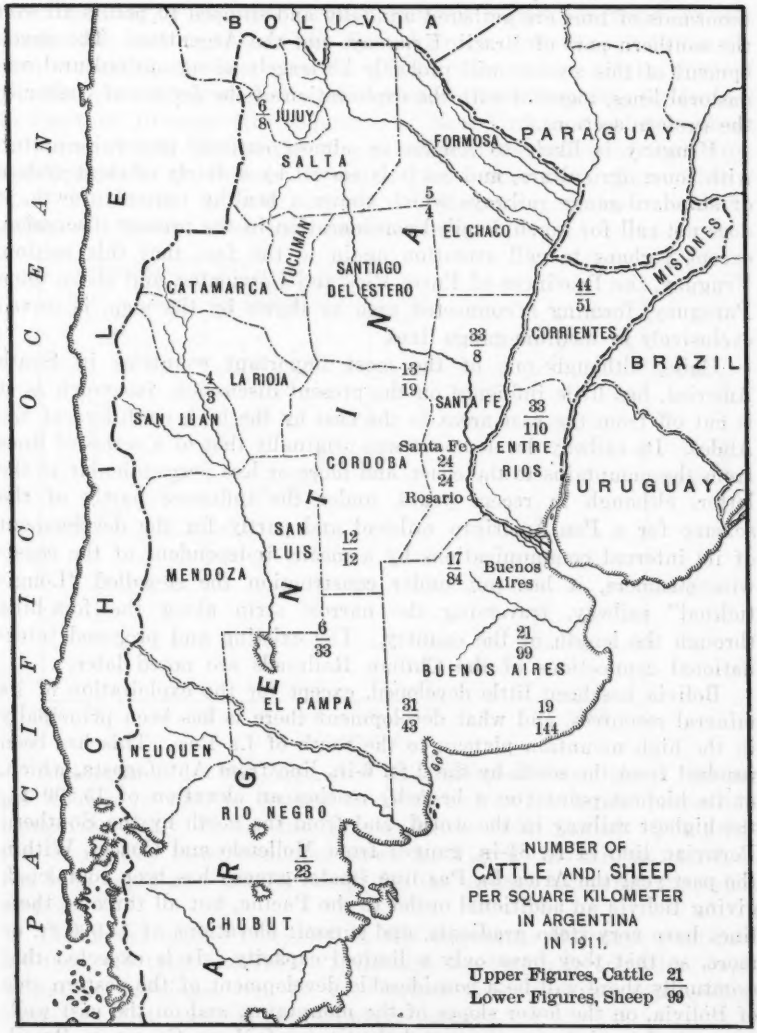

BOLIVIA
PARAGUAY
BRAZIL
URUGUAY
PACIFIC OCEAN
ATLANTIC OCEAN
ARGENTINA
6/8 JUJUY
SALTA
FORMOSA
5/4 EL CHACO
MISIONES
TUCUMAN
SANTIAGO DEL ESTERO
CATAMARCA
44/51
33/8 CORRIENTES
4/3 LA RIOJA
13/19
SANTA FE
33/110 ENTRE RIOS
SAN JUAN
Santa Fe
CORDOBA
24/24
Rosario
SAN LUIS
12/12
MENDOZA
17/84 Buenos Aires
6/33
21/99 BUENOS AIRES
EL PAMPA
31/43
19/144
NEUQUEN
RIO NEGRO
1/23
CHUBUT
NUMBER OF
CATTLE AND SHEEP
PER SQUARE KILOMETER
IN ARGENTINA
IN 1911.
Upper Figures, Cattle 21
Lower Figures, Sheep 99

FIG. 5.

The principal products of the three southerly States of Brazil, namely: Parana, Sta. Catharina, and Rio Grande do Sul, are timber, some agricultural products, and the yerba maté, of which latter thousands of tons are gathered annually and shipped to points all over the southern part of Brazil, Uruguay, and the Argentine. The development of this section will probably be largely along agricultural and pastoral lines, together with the exploitation of the forests of timber in the western sections.

Uruguay is likely to remain an almost entirely pastoral country, with some agriculture, and, as it is served by a fairly efficient system of standard-gauge railways which shows a healthy normal growth, it does not call for much detailed consideration in the present discussion, except perhaps to call attention again to the fact that this section, Uruguay, the Provinces of Entre Rios and Corrientes, and above them Paraguay, forming a connected area as shown by the map, is served exclusively by medium-gauge lines.

Chile, although one of the most important countries in South America, has little influence on the present discussion, inasmuch as it is cut off from the vast areas to the east by the high cordillera of the Andes. Its railway development was originally that of a series of lines from the mountains to the coast, and more or less perpendicular to the latter, although in recent years, under the influence partly of the scheme for a Pan-American railroad and partly for the development of its internal communications by a means independent of the coastwise steamers, it has put under construction the so-called "Longitudinal" railway, traversing the narrow strip along the foot-hills through the length of the country. The existing and proposed international connections of the Chilian Railroads are noted later.

Bolivia has been little developed, except for the exploitation of its mineral resources, and what development there is has been principally in the high mountain plateau to the south of La Paz. This has been reached from the south by the 2 ft. 6-in. line from Antofogasta, which, at its highest point (on a branch) reaches an elevation of 15 809 ft., the highest railway in the world, and from the north by the Southern Peruvian line (4 ft. 8½-in. gauge) from Mollendo and Cuzco. Within the past year the Arica-La Paz line (meter-gauge) has been completed, giving Bolivia an additional outlet to the Pacific, but all three of these lines have very steep gradients, and summit elevations of 15 000 ft. or more, so that they have only a limited capacity. It is expected that eventually there will be a considerable development of the eastern side of Bolivia, on the lower slopes of the mountains, and on the vast well-watered plains between them and the State of Matto Grosso in Brazil, and the outlet from this section of the country must inevitably be toward the Atlantic seaboard, either south through the Argentine, which if longer is, though much easier country, eastward through Para-

guay and Brazil to Rio de Janeiro and Santos, or northward through the Valley of the Amazon.

All this vast section seems to be entering on an era of development fostered by an ever-increasing tide of emigration from Europe, comparable only to that of the United States, and when it is considered—to take only one example—that the commerce of the City of Buenos Aires is growing at a rate at which it will probably be doubled in less than 10 years (the commerce of New York is doubled in about 20 years) it will be seen that the question of the policy of the future development of the railways is one of great importance. Figs. 6 and 7 show the increase in trade, exports, and imports, and the areas under cultivation for the three most important cereals.

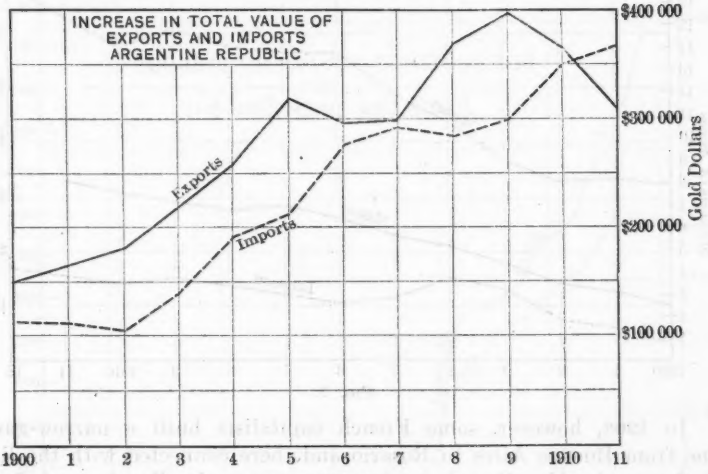


FIG. 6.

Immigration in the Argentine has increased from about 100 000 per annum in 1903 to 300 000 in 1910, by far the larger proportion being from Italy, Spain, and France, in the order named. The progress of railway construction is shown by Fig. 8. As will be noted, it has been rapid and steady, except for the setback following the crisis of 1890.

The latest official statistics of the Argentine Railways published are those for 1909, but Table 27, showing the length in operation, capitalization, earnings, etc., of the various lines for the year ending December 31st, 1912, is approximately correct.

The four broad-gauge lines are by far the most important. They have been built, and are controlled, by British interests. These lines radiate from the City of Buenos Aires, and have been most influential in the development of the country. They are so strongly intrenched

that they are a most important factor to be kept carefully in mind in any consideration of the question of gauge, as it seems entirely improbable that their owners and managers can be shown that there would be any benefit in changing.

The narrow-gauge lines of the Argentine have been principally developed in the section north of Rosario and Cordoba, and, until within comparatively recent times, partook more of the character of small local lines than of a connected system of trunk lines, none of them having reached the National Capital.

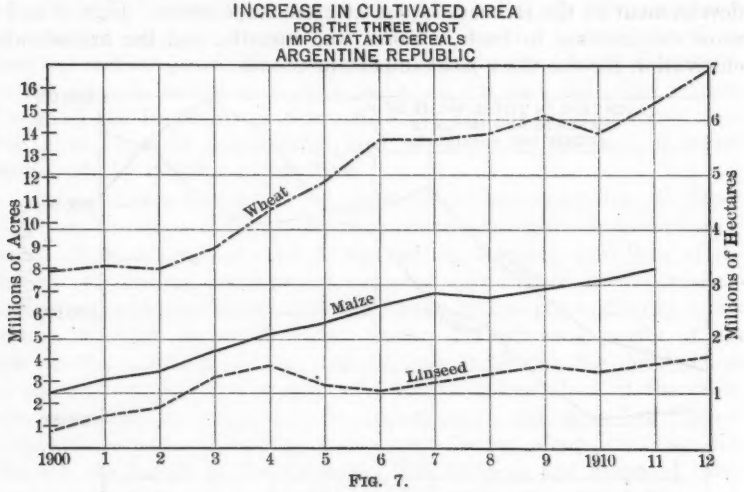


FIG. 7.

In 1908, however, some French capitalists built a narrow-gauge line from Buenos Aires to Rosario and there connected with the lines of the much older French company operating the Province of Santa Fé Railway reaching to Resistencia at the confluence of the Paraguay and Alto Parana, and connecting at Santa Fé with the Government lines through Tucuman to La Quiaca on the Bolivian border.

The Central Cordoba system, which had been formed by uniting some three or four small lines between Rosario, Cordoba, and Tucuman, with headquarters at Cordoba, decided about this same time to build an extension to Buenos Aires from Rosario, and thus connect its system with the National Capital, this line being opened to public service only about a year ago, and the headquarters of the system transferred to Buenos Aires. The Central Cordoba connects with the other part of the Government system, the Argentino del Norte, and also with the Central Norte at Tucuman.

The narrow-gauge lines are thus divided now into three groups: The French lines radiating in three directions from Buenos Aires and

extending north along the Parana up to Resistencia; the Central Cordoba starting from Buenos Aires and passing through Rosario and Cordoba to Tucuman, and the Government lines reaching all the capitals of the northern and Andean provinces and scattered along the foot-hills of the mountains between San Juan and Bolivia, connecting at Cordoba and Santa Fé with the other lines and thus having through communication with Buenos Aires.

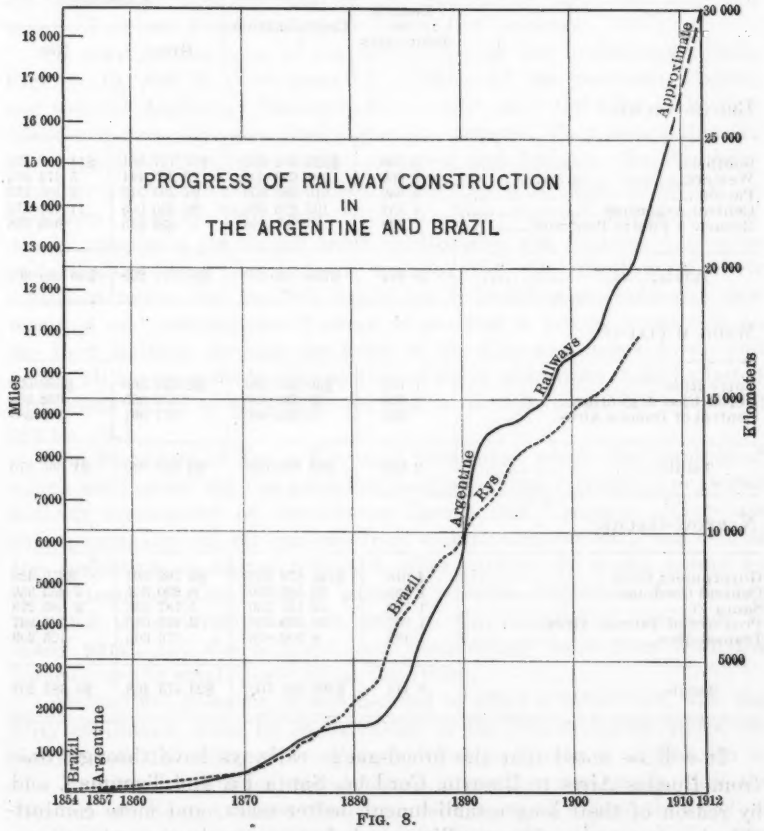


FIG. 8.

These conditions on the narrow-gauge lines must be kept well in mind, as they are just now emerging from a status as more or less isolated lines, doing a small local business, to that of a more or less coherent group of systems of trunk lines connecting the National Capital with the northern section of the country, and reaching nearly all the provincial capitals. As local lines doing a small business, the gauge in itself mattered little, but now, with the greatly increased

business that is coming to them, and the growing necessity of running through express trains at fairly good speeds, they find themselves handicapped by reason of this very increase in business, because they have not the facilities for doing it properly.

TABLE 27.

	Length, in kilometers.	Capitalization.	Earnings.	
			Gross.	Net.
Broad-Gauge.				
Southern	5 608	$220 503 600	$27 127 561	$11 311 439
Western	2 669	101 959 200	12 225 441	5 474 275
Pacific	5 342	216 086 900	24 405 713	9 185 113
Central Argentine	4 751	197 276 600	26 360 005	11 481 579
Rosario & Puerto Belgrano	794	30 957 100	553 575	903 958
Totals	19 164	$766 783 400	$90 672 295	$38 356 364
Medium-Gauge.				
Entre Rios	1 175	$30 391 700	$2 379 390	$908 096
North East Argentine	1 074	29 538 100	1 609 335	622 090
Central of Buenos Aires	269	8 942 800	977 091	305 389
Totals	2 518	$68 872 600	$4 965 816	$1 835 575
Narrow-Gauge.				
Government lines	4 018	$121 872 900	$6 292 069	$359 428
Central Cordoba	1 935	70 525 000	8 220 351	2 253 352
Santa Fé	1 709	42 131 700	5 787 433	2 085 278
Province of Buenos Aires	1 267	39 399 300	2 497 010	527 847
Transandine	185	8 902 800	676 605	125 299
Totals	9 114	$282 831 700	$23 473 468	$5 383 204

It will be noted that the broad-gauge railways have through lines from Buenos Aires to Rosario, Cordoba, Santa Fé, and Tucuman, and, by reason of their long establishment, better track, and more comfortable, because easier-riding, rolling stock, have up to the present monopolized the through passenger business to and from these points. In order, therefore, to get their share of this business, the narrow-gauge lines will be forced to make considerable improvements in their service, starting with the physical condition of their roadbed and track, and working up and even then, of course, can never compete on even terms.

It is often pointed out, in discussing the gauge, that these narrow-gauge lines have done very well, that their capitalization is small and, therefore, there is no reason to change, but, as will be shown later, it seems to the writer that they must look forward to the growth of the country and be prepared to meet it by modern economic means of transport or be crowded out. By the time they have provided adequate terminal facilities in the large cities, their capital expenditures will not be so much less than that of the broad-gauge lines, and they can never, of course, handle the same amount of business.

To convey some idea of the importance of the broad-gauge lines, Figs. 9, 10, and 11 show parts of the lines of the Southern, Pacific, and Central Argentine Railways, where they enter the City of Buenos Aires over expensive steel and masonry viaducts. They have extensive and expensive terminals, both passenger and freight; the Southern has four tracks for some distance out of the city. Fig. 12 shows the Southern Railway Passenger Station. The Central Argentine has nearly completed its second track to Rosario; the Central Argentine and Western have plans under way for the electrification of their suburban zones; the Central Argentine is building an extensive new terminal and station; the Western is building a tunnel connection to the Port Railway through the heart of the City of Buenos Aires, and nearly all have a considerable portion of their main lines stone-ballasted with stone at $2 to $3 (gold) per cu. m. f. o. b. cars ($1.50 to $2.50 per cu. yd.).

The Provinces of Entre Rios and Corrientes, within the confines of which are located the two principal medium-gauge railways, are almost entirely surrounded by the Rivers Parana and Uruguay. They are thus practically cut off entirely from communication with the rest of the Argentine, or indeed with any other part of the world, except by water transportation. Their isolated position, therefore, has rather retarded their development, in spite of the richness of the lands in many parts, and the railways have been merely local lines from the interior to the small ports along the rivers.

About 1906, however, it was decided to effect a connection with the City of Buenos Aires by an extension of the line to Ibicuy and a car ferry across the delta of the Parana River to Zarate, a small town in the Province of Buenos Aires. Here connection was made with the Central of Buenos Aires Railway, then a rural steam tramway of medium-gauge, over which running rights were obtained into the City of Buenos Aires, though at a point a fairly long distance out from the center. This through line was opened to public service only about 4 years ago.

At about the same time, also, the Central Railroad of Paraguay, which at that time was a broad-gauge line running from Asuncion, the capital of that country, to Villa Rica in the interior, was changed

to medium-gauge and extended to Encarnacion, opposite Posadas in Corrientes, and another car ferry established there, so that by these two car ferries, through train service between the City of Buenos Aires and Asuncion, through the Provinces of Entre Rios and Corrientes is now possible, thus putting all this section in close touch with Buenos Aires and the outside world.

It will be seen, therefore, that the whole railroad situation in the Argentine has undergone a radical change during the past 5 years or so. Previous to that time, the four large broad-gauge systems controlled by British interests, which in many respects were identical, dominated almost exclusively the transportation of the country, and the lines of other gauges were of little importance. Now the medium-gauge lines are not only linked together, but are all (except the Central of Buenos Aires) controlled by mutually friendly interests with a through route some 900 miles long from Buenos Aires to Asuncion. The narrow-gauge lines are linked up into not more than three systems, all of which are working together in harmony and may come under a single control. They reach the northern section of the country and the one in which the great development of the next 20 years is to take place. The question is, shall this development be by means of a system of narrow-gauge lines, can such a system offer adequate transportation facilities, and can it offer competition to the broad-gauge lines, should they decide to extend their lines into this section, and with river transportation on the Parana?

There is also to be considered the question of international connections, and the present and rapidly increasing interest in South America, both by Europe and North America, makes this important. To the writer the idea of a Pan-American Railroad as a continuous trunk line for the transportation of through freight and passengers from the United States and Canada to the extreme south, has always seemed almost chimerical. The variation in gauges, as will be seen, is an important obstacle, and such a line can never hope to compete for long-distance transportation through a tropical climate and across the vast and lengthy stretches of the most rugged mountains in the world, with the facilities offered by ocean steamers.

It is easy, of course, to foresee that at some time in the future, which seems now not a little distant, the various lines of railway will be linked together, and that there will be continuous connections. This, however, will not be, at least in the writer's estimation, until there is a local demand and local business sufficient to warrant the construction of the various constituent parts. It is also entirely probable that such a line will be useful and will be used for international communications, between any two or more countries within reasonable limits, and for this reason uniformity of gauge should be advocated throughout the continent and through the Central American zone,

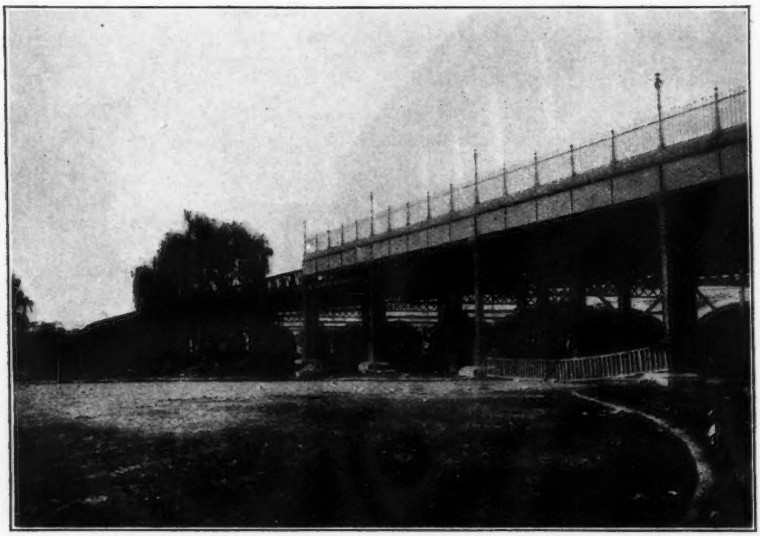


FIG. 9.—OLD STEEL VIADUCT, CENTRAL ARGENTINE RAILWAY, ENTERING BUENOS AIRES.

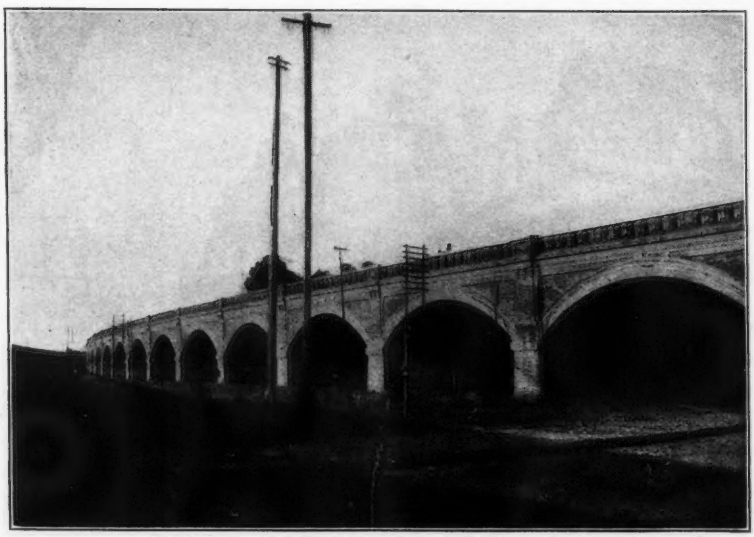
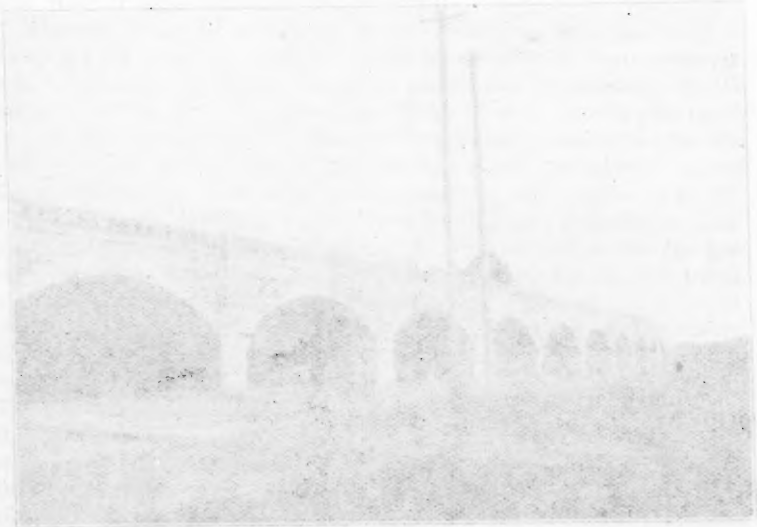


FIG. 10.—MASONRY VIADUCT, PACIFIC RAILWAY, ENTERING BUENOS AIRES.



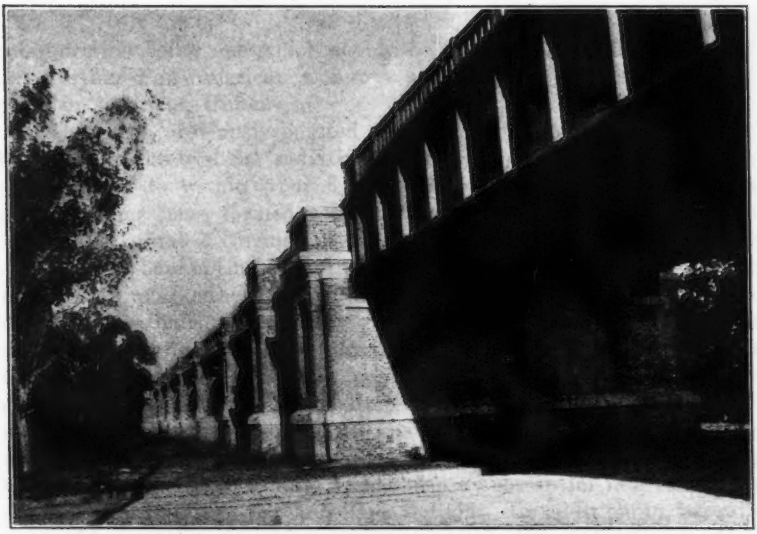


FIG. 11.—STEEL BRIDGE AND MASONRY VIADUCT, CENTRAL ARGENTINE RAILWAY, ENTERING BUENOS AIRES.

FIG. 12.—SOUTHERN RAILWAY PASSENGER TERMINAL STATION, BUENOS AIRES.

this question being one which should be most energetically pressed at any further Pan-American Congresses and by the representatives of the Pan-American Union.

It has been previously pointed out how, in the Argentine, the four big broad-gauge systems radiating from Buenos Aires dominated the situation up to within 5 or 6 years ago, and how, since then, the other systems have formed coherent groups with connections to the National Capital. During the same period, also, an international syndicate of bankers has acquired control of the railway lines in the three States of Southern Brazil, and a substantial interest in those in the southerly half of the State of São Paulo. This same syndicate also has operating control over three lines in the Argentine, the Rosario á Puerto Belgrano, 5 ft. 6-in. gauge, the Central Cordoba, meter-gauge, and the Entre Rios, medium-gauge, it has established a community of interest with the North East Argentine Railway, which connects the Entre Rios Railway with the Paraguay Central Railway, which latter it also controls. It has also a substantial interest in the Antofogasta Railway, in Chile and Bolivia. In addition to its railways, this syndicate is taking great interest in the exploration, exploitation, and development of the countries served by its lines and those beyond into which they may be extended.

One of the lines controlled by these interests has already been pushed across the State of São Paulo, crossing the Alto Parana nearly 1 500 miles above its mouth, and, eventually, without doubt, will be pushed ahead through the State of Matto Grosso into Eastern Bolivia and to Sucre on the headwaters of the Amazon. Another line is projected along the boundary line between the States of Parana and Sta. Catharina, following the Valley of the Iguazu to its confluence with the Alto Parana, crossing the latter at a point some 500 miles below that of the line into Matto Grosso, just referred to, and connecting with the line now under construction eastward from Villa Rica on the Paraguay Central. This will give a through route from Rio, São Paulo, and all points in Southern Brazil to Asuncion, the capital of Paraguay, though there will be a break of gauge at the border. From Asuncion this line will probably be extended northwestward between the Pilcomayo and Paraguay Rivers toward Sucre.

There is another east and west line still farther south in Brazil from Porto Alegre to Uruguayana, a point on the River Uruguay opposite Paso de los Libres on the North East Argentine Railway, where connection can be made either to Santa Fé or Resistencia on the west side of the Parana, and from these latter places to all points in the northern section of the Argentine and to Bolivia. This latter route at present involves more or less delay, steamer transportation across the two rivers, and two breaks of gauge, but eventually this will be developed into a through route.

There is already through rail connection between Rio and Montevideo, with breaks of gauge, however, at São Paulo and at the border of Uruguay and Brazil. At least two schemes have been proposed for a short through line from Colonia, a point in Uruguay opposite Buenos Aires, to Rio, over which passengers can be carried in express trains at reasonably fast speeds, and thus shorten the length of the sea trip between the Argentine and Europe, and the time between Buenos Aires and Rio by nearly half. This, if it is ever realized, means a standard-gauge line through Southern Brazil, as the Uruguayan gauge is fixed and it would be out of the question to get the necessary degree of speed and comfort on any lesser gauge to compete with the fast, luxurious ocean steamers in which the trip can now be made and which are being improved every day. Perhaps present conditions hardly warrant the construction of this line as yet, but that it is a development of the not far distant future is hardly open to question.

There is, as is perhaps well known, one line which now crosses the Andes, giving direct rail connection between Buenos Aires and Valparaiso. This, however, involves a change at the break of gauge at Mendoza in the Argentine and at Los Andes in Chile, the Buenos Aires and Pacific is 5 ft. 6-in. gauge, the Transandine in the Argentine and Chile is meter-gauge, and the Chilean lines with which it connects are 5 ft. 6 in. Another connection is proposed in the south by the extension of the Southern Railway's (5 ft. 6 in.) line from Neuquen, and there is a proposal (which, however, it does not seem will be carried through in the immediate future) to build a line from Salta in the Argentine to Mejillones in Chile. Farthern north, the Antofogasta Railway has been building an extension southeastward from Uyuni toward Tupiza, and probably before long it will be connected with the lines of the Argentine Government (meter-gauge) at La Quiaca.

Turning now to the Argentine: The area west and south of Bahia Blanca, it seems, must inevitably be developed by the Southern and Western Railways, that is, by the 5 ft. 6-in. gauge, and the country north of Santa Fé and Cordoba is the part in which may be expected the most important developments of the future which will be of interest to this discussion.

The line of the Central Cordoba Railway from Cordoba to Tucuman, and the Government line from Tucuman by Salta to Embarcacion may be taken as marking the dividing line between the mountains and the vast Argentine plain. Between Santa Fé and Tucuman there are two lines, the more southerly being the Central Argentine broad-gauge, and the more northerly the Central Norte, meter-gauge, operated by the Government. It is the area north of this latter line and east of the line from Tucuman to Embarcacion, and known generally as the Gran Chaco, in which the development of new territory by new

lines will probably be most active in the Argentine. The topography of this region is almost as blank as the space shown on the map, and its character does not change for at least some distance north of the Pilcomayo in Paraguay and Bolivia. The distance from Santa Fé to the northerly border of this region is about 750 miles, and from Asuncion to Tucuman about 550 miles, and it is probably safe to assume that railways may be built at a minimum cost and in practically straight lines over almost any portion of it.

Santa Fé is a port at present available at lowest river stages for ocean-going steamers drawing 18 ft. of water, and this depth will probably be increased in the near future. It seems to the writer improbable that there will be a port for ocean-going steamers established north of this point, so that the commerce of this vast region must come to Santa Fé. It is true that there may be some competition by way of the rivers, but experience elsewhere has shown that this will not be a serious matter for properly equipped railroads, though it might be for lines any less efficient than those designed in the light of the best present knowledge of the art. This area must be served by railroads of some sort, they must be efficient if they are to pay and be able to compete with water transport.

The Argentine Government has commenced the development of this region by lines running more or less westward from points on the Parana River at Resistencia and Formosa, apparently with the idea of bringing the products from the interior to the river and thence by fluvial transportation to the coast, the merchandise and supplies returning the same way. This, the writer believes to be fundamentally wrong, and that this area might better be developed by a series of north and south lines from Santa Fé and points on the existing lines between Santa Fé and Tucuman, which will reach up into the north beyond the Pilcomayo and bring all this vast region in touch with Buenos Aires. Here it may perhaps be well to refer again to the importance of the City of Buenos Aires as a distributing point for all merchandise, its dominating influence over the whole country as the seat of the national government, and the absolute importance of proper and adequate means of transportation to reach it from all parts of the country.

A certain number of east and west lines between Brazil and the north, along the lines already being developed, and as already indicated, will be necessary of course for the interchange of products and for communication between the large populations which will, in all probability, occupy this area before another quarter of a century has passed, but the line of least resistance to and from the southern part of eastern Bolivia, western Paraguay, and the Chaco is by the most direct north and south routes between this section and Buenos Aires.

The exact details of the lines along which the future development of this area may proceed, however, are not of particular importance in this discussion. The outline of the general scheme which has been presented will at least tend to show its close resemblance to that of the United States during the time our great West was being opened up to agricultural developments. Knowing as we do the important role played by the railroads in this era, it seems obvious that a system of transporation less efficient than our own in moving bulk freight long distances at low cost would handicap the development of the great area in the Southern Hemisphere.

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THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

By J. A. L. Waddell, M. Am. Soc. C. E.

To be Presented April 15th, 1914.

Summary.

As bridges of exceedingly long span are needed these days in a number of places, it has become necessary to use alloys of high elastic limit and great ultimate strength, in order to lessen materially the dead load to be carried. Up to the present time, nothing more satisfactory or effective than nickel steel has been found; and even that alloy has not been developed to the limit of its practical possibilities. This is due solely to a reluctance on the part of manufacturers of bridge metal to take any special pains in its production.

The time has come to discover some alloy of steel (or some other metal) of great strength, which, in every detail, will be suitable for undergoing the manipulations required of bridge material during the various processes of fabrication.

The objects of this paper are:

First.—To show, for the different span lengths, what the weights of metal per linear foot of bridge would be, in both simple-truss and cantilever structures, when using metals of various elastic limits, ranging from that of ordinary carbon steel up to 100 000 lb. per sq. in.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Second.—To indicate the extreme practicable limits of span length for cantilever bridges constructed for the greater part of such materials.

Third.—To demonstrate the comparative economics of finished bridges involved in using metals of the several assumed elastic limits, for all reasonable excess costs of metal per pound, delivered at site, as compared with carbon steel structures.

In the diagrams which form a part of the paper, Figs. 5 and 6 show the weights of metal per linear foot of structure for double-track railway bridges of simple-span and cantilever construction, beginning with short spans and extending to the limits of utility. Directions are given in the text for finding the weights of metal per foot for bridges having any number of tracks and for those designed with other live loads than those adopted in the preparation of the paper. From these diagrams and from the text it will be seen that the possible limits of main span length for cantilever bridges vary from some 2 000 ft. for carbon-steel structures to about 3 500 ft. for those built of metal having an elastic limit of 100 000 lb. per sq. in.

Figs. 10 to 21, inclusive, are diagrams showing the costs of metal erected per linear foot of bridge, in both simple spans and cantilevers, for the different elastic limits assumed. There is a curve for each excess pound price of metal work, delivered at site, above that of carbon steel; and these excesses vary by ½ cent. per lb., and extend always slightly beyond the greatest increase that need be considered. On each diagram there is shown, by a line heavier than the others, the cost of erected metal per foot of structure for carbon-steel bridges under the conditions assumed, so as to afford a means of instantaneous comparison of costs between any all-carbon-steel bridge and the corresponding structure built chiefly of the metal to which the diagram pertains.

Figs. 22 and 23 show the comparative economics of all the alloy steels (or other metals) considered, under the assumption that their pound prices for material delivered at the bridge site are the same.

In his paper on "Nickel Steel for Bridges",* the writer showed the results in bridge construction obtainable by the use of nickel steel, and although the limits of attainment indicated therein have not yet

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

been reached by American bridge builders, because the metal manufacturers are not willing to guarantee an elastic limit of 60 000 lb. per sq. in. for that alloy, nevertheless, since its publication, a number of important structures have been built of nickel steel, and the manufacturers have indicated to the engineers a willingness to increase their set elastic limit of 50 000 lb., provided they be satisfactorily compensated. The obstacle in the way of adopting a 60 000-lb. elastic limit is the fact that there is no market for the rejected material. The writer is of the opinion, however, that, after a little experience, the manufacturers would have no more trouble in furnishing the alloy with an elastic limit of 60 000 lb. than in furnishing ordinary carbon steel with an elastic limit of 35 000 lb. The best practicable solution of the difficulty is an agreement by the purchaser to pay the manufacturer a fair compensation for his loss by rejections. The writer's reason for thinking that no great trouble will be encountered in manufacturing nickel steel with a minimum elastic limit of 60 000 lb. is that the inspection sheets for the Free Bridge over the Mississippi River at St. Louis, engineered by the late Alfred P. Boller and Henry W. Hodge, Members, Am. Soc. C. E., seldom show an elastic limit materially less than that, although the requirement fixed by arrangement with the manufacturers was only 50 000 lb.

It is the writer's intention, on the first opportunity that presents itself to him for building a bridge of very long span, to construct it of mixed nickel and carbon steels, placing the alloy wherever it can be used to advantage, and arranging, if possible, with the manufacturers for a minimum elastic limit of 60 000 lb., so as to show actually what can be accomplished by adopting nickel steel of that strength, and complying with the specifications given in the before-mentioned paper on "Nickel Steel for Bridges".

A condition which at present militates seriously against the use of nickel steel in bridge building is that the manufacturers ask for it an additional price of some 2 cents per lb., as compared with ordinary carbon steel, although but little more than one-half of that would be a sufficient excess price. This, undoubtedly, is because structural steel manufacturers naturally object to fundamental innovations, preferring to follow lines of least resistance; but, as a number of very long steel spans are contemplated for the near future, they will certainly be forced sooner or later into using high-alloy steels.

Again, it has been stated authoritatively to the writer by Mr. T. L. Willson, the eminent Canadian authority on thermo-chemistry, that ferro-nickel, containing 10% of nickel and nearly 90% of iron, can be furnished with a good profit at 2 cents per lb. for adding to the molten carbon steel in the manufacture of nickel steel, and that no trouble would be involved in burning out all the impurities contained in the said ferro-nickel. This would make the nickel content in the alloy cost about 10 cents per lb. instead of 30 cents, as assumed in the before-mentioned paper. The economics involved by such use of ferro-nickel will be treated hereinafter.

During a recent conversation with two of the largest European producers of nickel, the writer was told that it would be practicable for them to furnish pure nickel in large quantities for the purpose of manufacturing bridge material and to sell this pure nickel at about 20 cents per lb.; but they dropped the hint that they were in no hurry to do so, as the demand for that metal to-day is in excess of the supply, and the main obstacle in the way of large production is likely to be a scarcity of satisfactory nickel ores.

When in France in 1909, the writer's attention was called to the fact that certain metal manufacturers in that country were making, in melts of 5 tons and less, a purified carbon steel, for which rather astonishing claims, in regard to great ultimate strength, high elastic limit, and general suitability for the manufacture of bridges, were made; and on his investigating the matter by both interview and correspondence, he was convinced that these claims might, at least partly, be justified by performance. Thereupon, having some spare time, he prepared an economic study of the possibilities for utilizing such purified steel in bridges, using in his calculations French units, prices, and other conditions, and published the results in French,* under the title, *"Étude Économique de l'Emploi de l'Acier au Carbone à Grande Résistance, pour la Construction des Ponts."*

The excess cost of this purified steel, as compared with ordinary carbon bridge steel, was claimed to be a little less than 1 cent per lb. for the manufactured bridges; and the investigation showed the economics of its use in bridge building for the mean and the extreme conditions of the French metal market, and for a number of elastic limits varying

* *Le Génie Civil*, August 7th, 1909.

from 30 to 45 kg. per sq. mm., the value for French carbon bridge steel being 24 and that for the writer's specified nickel steel 42.5 kg. per sq. mm. The result of the study indicated that there was no advantage for the 30-kg. elastic limit; none for short spans but a small one for long spans with a 35-kg. elastic limit; a decided saving for all cases with a 40-kg. limit; and a wonderful economy for the 45-kg. limit, the highest elastic limit claimed by any of the French manufacturers.

The writer had hoped that that paper would give an impetus in France (and perhaps elsewhere also) to the manufacture of bridges of purified steel; but, up to the present time, he has not heard of any such development. It is either that the claims of the manufacturers have not been justified by performance, or that the same conditions of inertia and *laisser aller* exist in Europe, in respect to innovations in the manufacture of new bridge metal, as govern in America in relation to the adoption of nickel steel for bridges.

As the future development of America will necessitate the building of many very long-span bridges, it is almost a necessity that there be found an alloy of steel of great strength, high elastic limit, workable under all necessary manipulations in the shops, and of moderate cost. Such an alloy is not going to be discovered by accident, but only by a lengthy and exhaustive series of experiments, laid out systematically in advance, and modified from time to time as knowledge of the subject is accumulated. These experiments should be performed where small melts of steel are readily procurable, and where the experimenter will not be unduly delayed by the metal manufacturers. These conditions are found in France—for there one can procure single ton melts, and possibly smaller ones, if necessary. Arrangements could be made with the mills for rolling into plates and shapes the metal from the various melts without delay; and the experimenter could have at his disposal, not only a first-class testing machine, but also a bridge shop, where all the usual manipulations of the metal might be made. By experimenting at first on different kinds of alloy materials and later on mixtures thereof, one eventually should discover some satisfactory combination that would afford a suitable material of great strength at comparatively moderate cost. The more elaborate and thorough the experiments, the greater the probability of discovering a truly suitable steel of exceedingly high resistance at a reasonable expenditure for production and manufacture.

Such a series of experiments would be expensive, costing fully $100 000, and possibly twice that amount; but the saving in cost on one big bridge alone might easily exceed the entire expenditure. The money required for such experiments could not be obtained from bridge manufacturers, because it is not to their interest to inaugurate changes; but it might be secured from some millionaire philanthropist or from a group of capitalists who contemplate the construction of some great steel bridge.

To show the possibilities for long-span bridge construction by the use of such superior alloys of steel, and the economics thereof, is the object of this paper.

The basis of the following investigation is a mass of diagrammed and tabulated data concerning weights of metal in simple-span and cantilever bridges of carbon steel, up to a limit of 600-ft. spans for the former and 1 800-ft. main openings for the latter, accumulated by the writer and his firm during the last quarter of a century, and the weights of nickel-steel bridges and of mixed nickel-steel and carbon-steel bridges computed by the writer in the preparation of his before-mentioned paper on "Nickel Steel for Bridges". Because the said diagrammed weights of metal per linear foot of span in simple-truss bridges are limited to lengths of 600 ft. it has been found necessary, in preparing this paper, to extend them to 1 000 ft. by making actual calculations of stresses, sections, and weights of metal for several long spans, using the various kinds of steel assumed. From these recorded and specially computed weights, and by the succeeding formulas of reduction and extension, have been prepared the various diagrams of weights of metal per linear foot of span contained in this paper.

Many preliminary and tentative diagrams of curves were constructed, but, for considerations of space and cost, they are not included herein. These individual diagrams, prepared for each class of steel and each type of bridge, divided the weights into those for "Floor System"; "Lateral System"; "Trusses"; and "On Piers"; but in the selected diagrams only the total weights of metal per linear foot of span for a combination of all the parts of the structures are recorded, as these quantities constitute the results sought, and to be used in the succeeding economic investigation. Near the end of this paper, however, are given some tables from which can be found the approximate values of weights of metal per linear foot of span for "Floor

Systems"; "Lateral Systems"; and "On Piers"; and from these and the diagrams, Figs. 5 and 6, can be computed the weights for "Trusses".

The weights given for bridges of carbon steel are based on the standard specifications for designing contained in the writer's "De Pontibus". They are quite accurate up to the before-mentioned limits of 1 000 ft. for simple spans and 1 800 ft. for the main openings of cantilever bridges. Beyond these limits (shown on the diagrams by dotted lines) they are to a certain extent conjectural, although, in all probability, not far from correct, because of the thorough consideration which has been given to all factors influencing their values.

The weights for steel having an elastic limit of 60 000 lb. per sq. in. are those obtained for the diagrams of the paper on "Nickel Steel for Bridges", and are based on the specifications contained in that paper. They are quite accurate up to the before-mentioned limits of 1 000 ft. for simple-truss spans and 1 800 ft. for the main openings of cantilever bridges; and a comparison of the curves for cantilevers of openings from 1 200 to 2 600 ft., shown in Fig. 71 of the writer's Nickel Steel paper with those of Fig. 6 of this paper indicates, in general, an excellent agreement. In the former the curves beyond 1 800-ft. openings were determined by an assumed regularity of continuity, as explained in that paper, and in the latter they were computed by the use of Equation 19 of this paper.

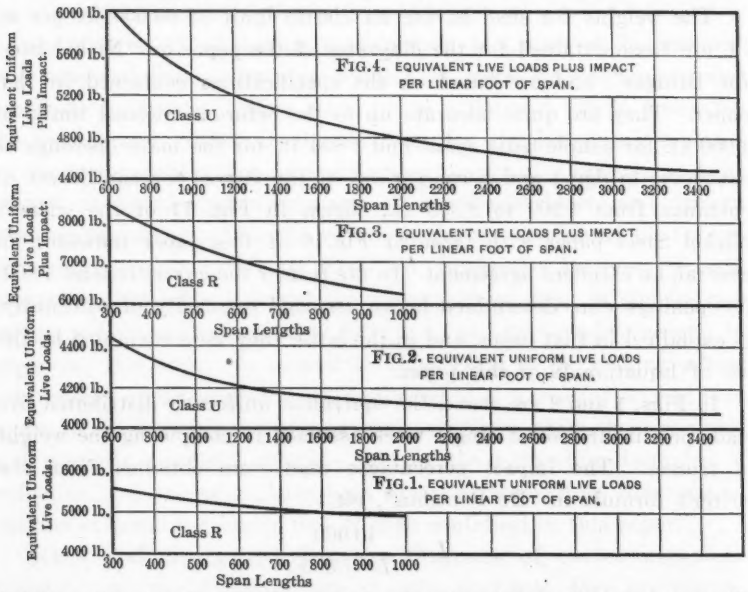
In Figs. 1 and 2 are shown the equivalent uniformly distributed live loads per linear foot of single track assumed in computing the weight of trusses. The impact percentages used were obtained from the writer's formula in "De Pontibus", *viz.:*

$$I = \frac{40\,000}{L + 500},$$

where I is the percentage of impact and L is the length, in feet, of the portion of the bridge which must be covered by the moving load in order to obtain the maximum stress on the member under consideration. Although it is true that the writer has evolved a more accurate formula than this, based on some lately made experiments on impact in railway bridges from moving loads, he has decided to adhere to the old formula so as not to necessitate the changing of the weight curves on which this investigation is primarily founded. Figs. 3 and 4 give a combination of the equivalent uniform live loads and the impacts therefrom. The loads obtained from these curves added to the dead loads

make the total loads per linear foot on the bridges, from which total loads the load ratios of reduction are found.

The following are the formulas of reduction used in passing from known weights of metal per linear foot of span in carbon-steel bridges to the corresponding weights in alloy-steel bridges. An observation of the nomenclature will show that the unaccented capital letters severally represent weights of metal per linear foot of span in carbon-steel bridges (or otherwise known weights of bridges of any kind of steel), and the accented capital letters, the corresponding weights for alloy-steel bridges (or otherwise the corresponding unknown weights of



bridges of some other kind of steel), also that the small letters severally represent linear dimensions of structures, the main exceptions being that capital R is used for reactions and small r for ratios.

Floor System.—

Let F = weight of metal per linear foot of span in the "Floor System" of carbon-steel bridges.

L = ditto for "Lateral System".

T = ditto for "Trusses".

P = ditto for "On Piers", including anchorage material in the case of cantilever bridges.

A certain portion of the weight of the floor system will vary inversely with the elastic limit of the steel, and the remainder will be invariable.

Let V = the variable portion,
and I = the invariable portion.

Then $F = V + I$ .....(1)

Let F' = the weight of metal per linear foot of span in the floor system of alloy-steel bridges,
and r (greater than unity) = the ratio of elastic limits of alloy steel and carbon steel,

Then $F' = I + \frac{V}{r}$ .....(2)

In heavy, double-track bridges, and especially those of long span, I will be approximately 0.35 F, and V approximately 0.65 F, hence

$$F' = 0.35\ F + \frac{0.65\ F}{r} = F\left(0.35 + \frac{0.65}{r}\right) \text{ .....(3)}$$

In dealing with spans of greater length than any of those yet actually computed, it must not be forgotten that the increasing width of structure will augment the weight of the floor-beams and, consequently, the weight of metal per linear foot of span for the floor system. In case of double-track bridges, an economy can be effected by widening the cantilever arms and the anchor arms uniformly from ends to supporting pier; but it is probable that motives of policy would lead the projectors to construct exceedingly long spans so as to carry more than two tracks.

Lateral System.—

Let l_1 = length of span at which it pays to begin to use high steel for the laterals beyond the ends of l_1, it being assumed that the weight of laterals is uniform over the entire length, l_1, or, in other words, that minimum sections are used therein throughout;

R_1 = wind reaction at end of l_1;

R = wind reaction at end of span l;

r_w = ratio (greater than unity) of R and R_1;

L_1 = weight of carbon steel per linear foot for lateral system over the length, l_1;

L'_2 = weight of mixed carbon and alloy steels per linear foot of span at end of span, l:

$$\text{Then } L'_2 = L_1 \left(0.3 + 0.7 \frac{r_w}{r}\right).$$

Let L'_a = average weight of metal per linear foot for entire span, l,

$$\text{Then } L'_a = \frac{1}{l} \left\{ L_1 \, l_1 + \frac{L'_2 + L_1}{2} (l - l_1) \right\} \ldots\ldots\ldots (4)$$

Should L'_2 figure less than L_1, it shows that near the ends of the span minimum sections of the high steel must be used and that L'_a will equal L_1.

In passing beyond the limits of actually figured spans, when computing the weights of metal in lateral systems, it must be remembered that, as just explained for the floor system, the weight per foot is increased, not only because of the greater span length but also because of the greater span width. As a rule, it may be stated that, for any very long span (the length thereof remaining constant), the effect of increasing the width between central planes of trusses $n\%$ is to increase the weight of metal in the lateral system about $\frac{n}{2}$ per cent.

Trusses.—In respect to the weight, T, of metal per linear foot of span for trusses of carbon steel, the following equation may be used:

$$T = K + T_1 + C_c + C_w \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where K is the portion of the total truss weight per linear foot which is independent of the quality of the metal and of the stresses; T_1 is that of the main portions of the tension members and of their details that are directly affected by the stresses; C_c is that of the main portions of the compression chords and inclined end posts and their details that are directly affected by the stresses; and C_w is that of the main portions of the compression web members.

From experience in designing large bridges it may be stated that, as an average,

$$K = 0.2T$$
$$T_1 = 0.3T$$
$$C_c = 0.3T$$
$$C_w = 0.2T$$

Both T_1 and C_c (and consequently their sum) will vary inversely with the elastic limit of the metal; but C_w, on account of the influence of the ratio of strut length to least radius of gyration, will not vary in that ratio. As an approximation it may be assumed that, in passing from any grade of steel to a higher grade, if, as before, r (greater than unity) is the ratio of the elastic limits of the two metals,

$$C'_w = \frac{1}{2} C_w \left(1 + \frac{1}{r}\right) \quad \ldots\ldots (6)$$

$$\text{and } C'_c = \frac{C_c}{r} \quad \ldots\ldots (7)$$

Substituting these values in Equation 5, we have

$$T' = K + \frac{1}{r}\left(T_1 + C_c\right) + \frac{1}{2} C_w \left(1 + \frac{1}{r}\right) \quad \ldots\ldots (8)$$

Substituting the values of K, T_1, C_c, and C_w in terms of T as previously given, we have

$$T' = T \left(0.3 + \frac{0.7}{r}\right) \quad \ldots\ldots (9)$$

In finding the new truss weight per linear foot for a higher steel, after computing it (as just indicated) for the direct effect of increased elastic limit, it must be corrected for the indirect effect, which is the changed total load per linear foot for trusses. This correction is made thus:

Find the sum of the live load, impact load, and dead load per linear foot of span, for the known truss weight, T, and then determine approximately the corresponding sum (on the basis of an assumed final value of T'_f) for the new truss weight. Let the ratio of these sums (less than unity) be r_1, then

$$T'_f = T' \ (0.3 + 0.7\ r_1) \quad \ldots\ldots (10)$$

where T'_f is the final value of the truss weight. Combining Equations 9 and 10 gives

$$T'_f = T \left(0.3 + \frac{0.7}{r}\right) (0.3 + 0.7\ r_1) \quad \ldots\ldots (11)$$

If the computed value of T'_f does not agree quite closely with its value adopted in determining the trial dead load, a new dead load is to be assumed, and the calculations are to be made afresh. The second attempt, in all probability, will give a sufficiently accurate agreement.

On Piers.—To find the new value, P', from the old value of P, the span length being unchanged, the following approximately correct equation may be used:

$$P' = P\left(0.6 + 0.4\,\frac{r_1}{r}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where r and r_1, respectively, are the ratios previously indicated for elastic limits and total loads per linear foot of span.

In extending a curve of simple truss weights of metal per linear foot of span beyond the limits of accurate computations, the following formulas may either be used directly or as a check, the character of the steel, of course, being unchanged. Assume first that the live and the dead loads per linear foot of span remain constant, and consider the effect only of longer spans and greater truss depths. Dealing first with the chords, some 85% of their weights of metal per linear foot of span vary directly as the moments of the total loads and inversely as the truss depths; but the moments vary as the squares of the span lengths, and the stresses are inversely as the truss depths. Again, the truss depths within short limits may, without serious error, be taken to vary directly as the span lengths. Such being the case, 85% of the weights per foot of the chords will vary directly as the span lengths, or

$$C' = 0.15\,C + 0.85\,C\,\frac{l'}{l} = C\left(0.15 + 0.85\,\frac{l'}{l}\right) \ldots (13)$$

where C is the chord weight per foot for the shorter span, l, and C' is the corresponding weight for the longer span, l'.

Let W and W' be, respectively, the weights of metal per linear foot of span in the webs of the two spans. About 75% of these will vary directly as the averages of all the live-load and dead-load shears on the spans, and these average shears vary almost directly as the span lengths. Again, the said 75% of W and W' will vary directly as the truss depths, and, therefore, as previously assumed, once more directly as the span lengths.

Combining these ratios will give the equation:

$$W' = 0.25\,W + 0.75\,W\left(\frac{l'}{l}\right)^2 = W\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\} \ldots (14)$$

but $T = C + W$,

and $T' = C' + W' = C\left(0.15 + 0.85\,\frac{l'}{l}\right) + W\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\}$.(15)

It is well known that in trusses with parallel chords and of economic depths the weight of the chords is equal to the weight of the web; but, in trusses with polygonal chords and having center depths less than the theoretically economic ones, as do those of all long-span bridges, the weight of the chords is much greater than that of the web. As a general average, we may assume that $C = 0.6\ T$, and $W = 0.4\ T$, hence

$$T' = 0.6\ T\left(0.15 + 0.85\frac{l'}{l}\right) + 0.4\ T\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\}$$

$$= T\left\{0.19 + 0.51\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\} \quad \ldots\ldots\ldots\ldots (16)$$

This value of T' is based on the incorrect assumption that the total loads per linear foot of span are the same for both spans under consideration, hence it requires a further modification, as follows:

$$T'_f = T'\ (0.2 + 0.8\ r_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

where T'_f is the final value of the weight of truss metal per linear foot of the longer span, and r_1 (in this case greater than unity) is the ratio of the total loads per linear foot.

Combining Equations 16 and 17, we have

$$T'_f = T\left\{0.19 + 0.51\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\}(0.2 + 0.8\ r_1) \ldots\ldots (18)$$

A test of this formula, on carefully computed curves of truss weights for simple spans of nickel steel from 600 to 1 000 ft. in length, shows that slightly undue prominence has been given to the invariable portion of the weights, and that the following modification of the formula will give more accurate results:

$$T'_f = T\left\{0.15 + 0.55\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\}(0.15 + 0.85\ r_1) \ldots (19)$$

This last formula, when tested on the truss weights of simple spans from 700 to 1 000 ft. in length for an elastic limit of 90 000 lb., gave exceedingly close results; hence it is proper to adopt it as the equation for extension of all truss weights for simple spans, and, inferentially, for those of cantilever bridges; in fact, it has been tested on some of the actually computed truss weights of cantilever bridges and found to give excellent agreement.

Attention is called to the semi-rational, semi-empirical character of these reduction and extension formulas. They are, in general, the result of long personal experience in the quick computation of metal

weights for bridges; but they have been modified slightly, as hereinbefore indicated, to agree with certain checks that have been made in this investigation. As far as practicable, the formulas of Equations 11 and 19 were used for checking each other; and the results of such checks were always satisfactory. For instance, if a curve of truss weights for one class of steel were used as a basis for finding, by Equation 11, the corresponding curve for another class of steel, the latter curve would be checked by starting from any desired point (generally where the weights of actually computed bridges cease) and passing, by using Equation 19, from one span length to another, 100 or 200 ft. greater, and continuing in this manner to the superior end of the curve.

Fig. 5 gives the total weights of metal per linear foot of span in simple-truss bridges with Class R live load. A glance at it shows the great saving in weight of metal obtained by using any of the alloy steels in preference to carbon steel. The difference is most apparent between the weights for $E = 50\,000$ lb. (the nickel steel that the metal manufacturers are willing to furnish) and those for $E = 60\,000$ lb. (the nickel steel used by the writer in his series of experiments some 8 or 10 years ago, and which alloy could be readily produced by the manufacturers through the expenditure of a little extra care and without any greater expense worth mentioning). The gradual reduction in the saving of metal with the increase of elastic limit is strikingly noticeable; and the conclusion may be drawn that, unless the extremely high-alloy steels can be obtained with only moderate increase in cost, there will be no economy in using them in simple-span bridges.

Fig. 6 gives the average total weights of metal per linear foot of span for cantilever structures having main openings of various lengths. The type of these bridges is the most common one (denominated "Type A" in "Nickel Steel for Bridges"), the live loads used being Classes R and S for the floor system and Class U for the trusses, as given in "De Pontibus." The proportional dimensions of this typical, through, cantilever bridge are as follows:

A main span, l, having a suspended span of three-eighths of l, and two cantilever arms each five-sixteenths of l in length, also two anchor arms of the same length as the cantilever arms. Any reasonable variation from these proportions would not change materially the average weight of metal per linear foot of span, as given by the curves

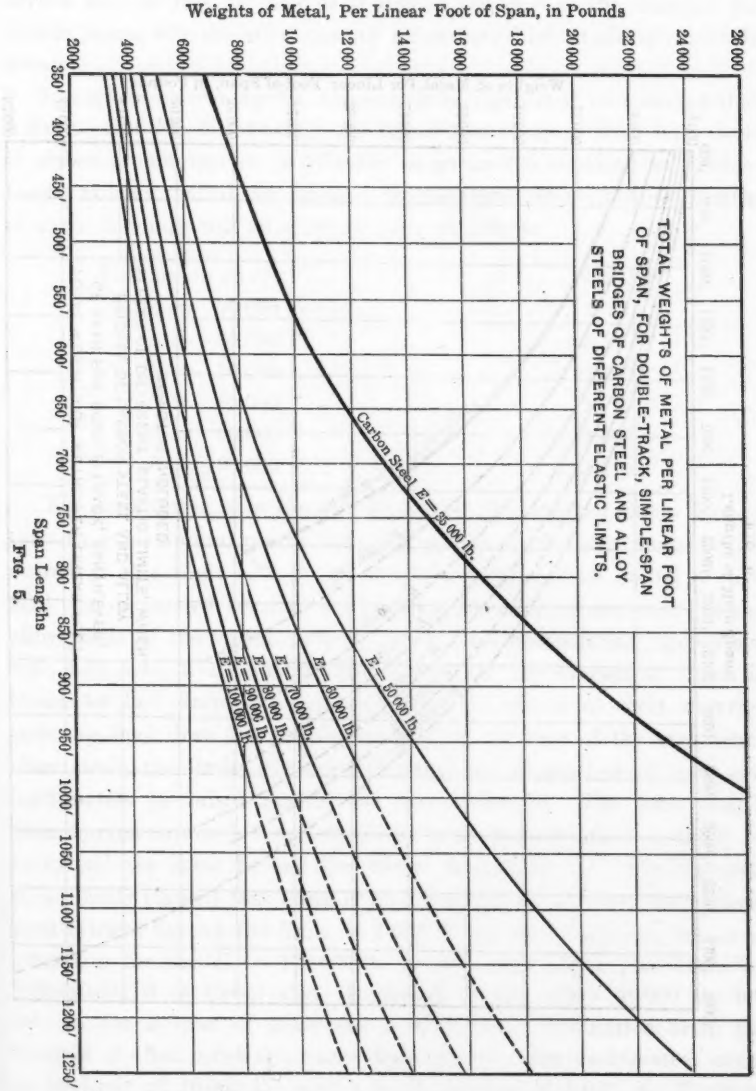

Weights of Metal, Per Linear Foot of Span, in Pounds
2000
4000
6000
8000
10 000
12 000
14 000
16 000
18 000
20 000
22 000
24 000
26 000
350'
400'
450'
500'
550'
600'
650'
700'
750'
800'
850'
900'
950'
1000'
1050'
1100'
1150'
1200'
1250'
Span Lengths
TOTAL WEIGHTS OF METAL PER LINEAR FOOT OF SPAN, FOR DOUBLE-TRACK, SIMPLE-SPAN BRIDGES OF CARBON STEEL AND ALLOY STEELS OF DIFFERENT ELASTIC LIMITS.
Carbon Steel E = 35 000 lb.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.

FIG. 5.

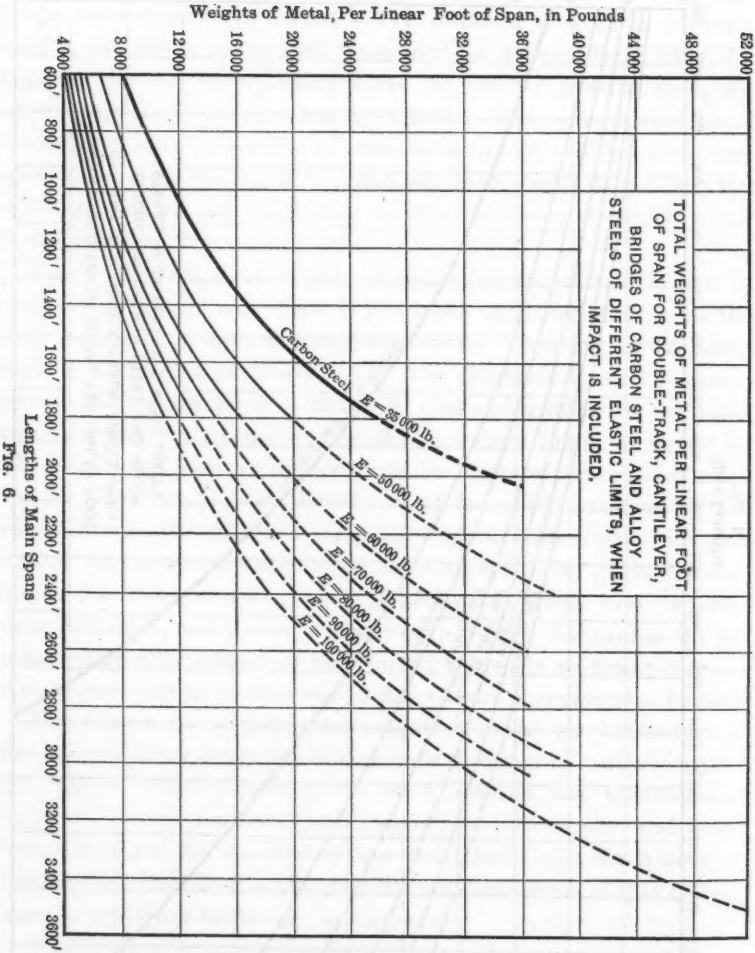

TOTAL WEIGHTS OF METAL PER LINEAR FOOT OF SPAN FOR DOUBLE-TRACK, CANTILEVER, BRIDGES OF CARBON STEEL AND ALLOY STEELS OF DIFFERENT ELASTIC LIMITS, WHEN IMPACT IS INCLUDED.
Weights of Metal, Per Linear Foot of Span, in Pounds
4 000
8 000
12 000
16 000
20 000
24 000
28 000
32 000
36 000
40 000
44 000
48 000
52 000
Carbon Steel E = 25 000 lb.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′ 3000′ 3200′ 3400′ 3600′
Lengths of Main Spans

FIG. 6.

on Fig. 6. In this figure the superiority of the alloy steels over the carbon steel is just as clear as it was in the case of the diagram for simple spans, but the advantage of using very high steels is evidently greater.

If (as can be seen by the diagram to be logical) it be assumed that a limit of 36 000 lb. of metal per linear foot of span is as high as it is either economical or practicable to go in the building of double-track, railway, cantilever bridges, the corresponding limiting lengths of main openings will be approximately as follows:

For carbon steel	.....	2 030 ft.
" $E =$ 50 000 lb. steel	.....	2 340 "
" $E =$ 60 000 " "	.....	2 590 "
" $E =$ 70 000 " "	.....	2 780 "
" $E =$ 80 000 " "	.....	2 910 "
" $E =$ 90 000 " "	.....	3 030 "
" $E =$ 100 000 " "	.....	3 140 "

The assumption of a limit of 36 000 lb. of metal per linear foot of span as a maximum, means that, for carbon steel, there would be required at this limit 4.35 lb. of metal to support each pound of live load that is carried (exclusive of impact allowance); and that for the alloy steels of the various elastic limits the corresponding figures are 4.37, 4.39, 4.40, 4.41, and 4.42 lb., respectively, the average of which is about 4.4 lb. From the appearance of the curves at their superior ends one may draw the conclusion that, in the case of the very high-alloy steels, the limit of weight of metal per linear foot of span can legitimately be raised beyond the said 36 000 lb. The more nearly these curves approach the vertical the more uneconomical would it be to extend the limit beyond the 36 000 lb. per lin. ft. For instance, it is plainly evident that there is no advantage in carrying the carbon-steel bridges beyond the limit of 2 000 ft. for main opening, but it is otherwise for the $E =$ 100 000-lb. curve. Continuing this curve by deflections, it is found that the weight would reach 46 000 lb. per lin. ft. for a span of 3 400 ft.; and that the inclination from the vertical at that point is greater than that for the carbon-steel curve at its limit of 36 000 lb. with a main opening of 2 030 ft. Perhaps, therefore, in the case of the highest steel herein considered, it would be more correct to assume the extreme economic or practicable limit

of main opening to be 3 400 ft. or even 3 500 ft. For this last length the average weight of metal per linear foot of structure shown by the $E = 100\,000$-lb. curve would be 52 000 lb., which means that it would require 6.38 lb. of metal to support each pound of live load, exclusive of the effect of impact. This seems to be an excessive quantity; nevertheless, it is conceivable that conditions might exist which would render it advisable to adopt this extreme limit of main opening, although, at such a length, a suspension bridge is undoubtedly cheaper than a cantilever structure.

If it be admitted that—as is maintained by some bridge engineers—in structures of very long span the impact of the live load on the main members of trusses is essentially nil, the practicable limiting length of main opening will be somewhat increased. Moreover, such a contention is not far wrong, because the latest experiments on impact from live loads on bridges show that the effect thereof on ordinarily long spans is much less than engineers in general have been assuming during the last two decades. That the impact ever reduces actually to zero is most unlikely; but, for openings of 1 200 ft. and greater, it is true that its amount is so small as to be negligible, in view of the fact that the live-load stresses on the main truss members will never be quite as great as they are computed, because, first, the trains on the two tracks never advance together so as to produce maximum web stresses; second, such trains are not likely ever to cover entirely the bridge or even any individual part of it, except, perhaps, the central span; and, third, it is improbable that any load of cars, unless they be ore or coal cars, will ever be uniformly full and loaded to the assumed limit.

In Fig. 7 are given the curves of weights for cantilever bridges of the same type and loading as those used in preparing the curves in Fig. 6, excepting that the impact on main members of trusses is assumed to be equal to zero. The curves in Fig. 7 begin at main openings of 1 200 ft. and extend to the greatest practicable limiting lengths of such openings. A comparison of Figs. 6 and 7 shows that, by ignoring impact on trusses, there is, on the average, a saving of some 700 lb. of metal per linear foot of span, for all spans and all kinds of steel. This difference, curiously enough, is comparatively uniform for all the curves and over their entire lengths, with a few exceptions. At first thought, one might imagine that the saving should be greater

for long spans than for short ones, but it must be remembered that, as the span length increases, the percentage of live-load impact diminishes. It is this fact which makes the difference under consideration so uniform.

Under the assumption that the limit of weight of metal is 36 000 lb. per lin. ft., the greatest practicable span lengths have been

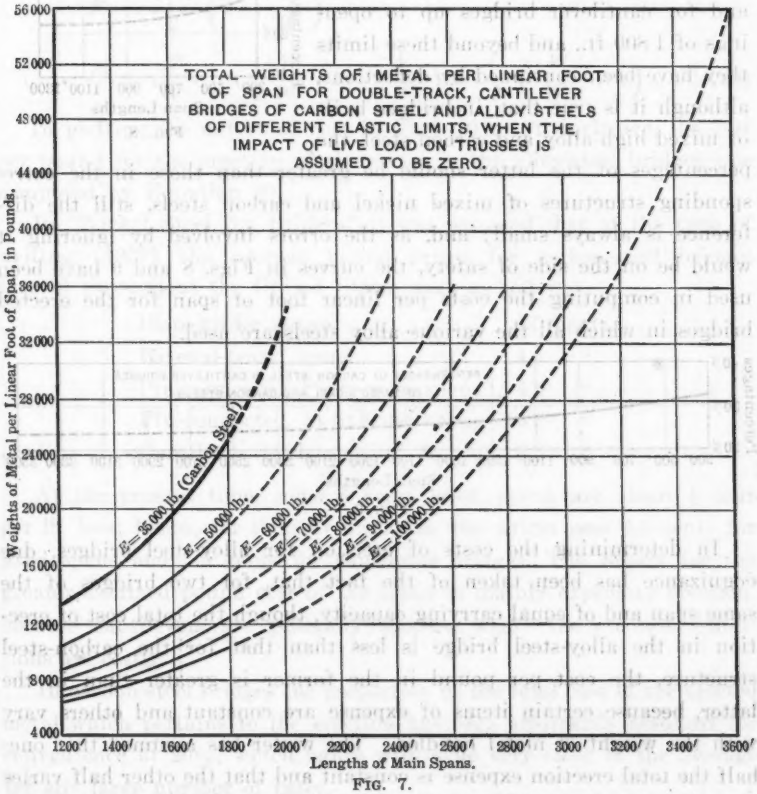


FIG. 7.

increased on the average only about 20 ft. by neglecting impact on trusses. Comparing the extensions of the curves for $E = 100\,000$ lb., it is found that, for an assumed limit of 52 000 lb. of metal per linear foot of span, the extreme practicable length of main opening has been increased only 25 ft. These various comparisons show that there is but little gain, either in economy or increase of practicable limit of opening, by neglecting the effect of impact; hence, in the economic

investigations which follow, the effect of impact has been duly considered.

In Figs. 8 and 9 are plotted, for both simple spans and cantilever bridges, the percentages of carbon steel in structures of mixed nickel and carbon steels. The curves are accurate for simple spans up to 600 ft. and for cantilever bridges up to openings of 1 800 ft., and beyond these limits they have been continued by deflections, although it is true that, in bridges built of mixed high-alloy and carbon steel, the percentages of the latter should be greater than those in the corresponding structures of mixed nickel and carbon steels, still the difference is always small; and, as the errors involved by ignoring it would be on the side of safety, the curves in Figs. 8 and 9 have been used in computing the costs per linear foot of span for the erected bridges in which all the various alloy steels are used.

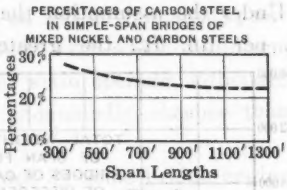


FIG. 8.

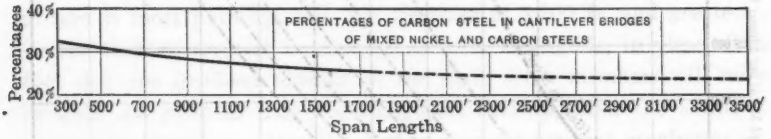


FIG. 9.

In determining the costs of erection for alloy-steel bridges, due cognizance has been taken of the fact that, for two bridges of the same span and of equal carrying capacity, though the total cost of erection in the alloy-steel bridge is less than that for the carbon-steel structure, the cost per pound in the former is greater than in the latter, because certain items of expense are constant and others vary with the weight of metal handled. The writer has assumed that one-half the total erection expense is constant and that the other half varies directly with the weight of metal. This is probably as accurate a division as can be assumed. On this basis was prepared, for the paper on "Nickel Steel for Bridges", the following mathematical statement:

Let W = weight of metal per linear foot of span in the carbon-steel bridge;

W' = ditto for the alloy-steel bridge;

C = cost per pound for erecting the carbon-steel bridge;

C' = ditto for the alloy-steel bridge;

F' = cost per linear foot for erecting the alloy-steel bridge;

then $C\ W$ = cost per linear foot for erecting the carbon-steel bridge.

$$F' = \frac{C\ W}{2}\left(1 + \frac{W'}{W}\right),$$

$$C' = \frac{F'}{W'} = \frac{C\ W}{2\ W'}\left(1 + \frac{W'}{W}\right) = \frac{C}{2}\left(\frac{W}{W'} + 1\right) \ldots\ldots (20)$$

In plotting the curves of cost in Figs. 10 to 21, inclusive, the cost per pound for the erection of the metal in the alloy-steel bridges was computed by Equation 20.

In "Nickel Steel for Bridges" it was assumed that at the time of writing (1907) the average pound prices for carbon-steel bridges erected throughout the United States were as follows:

Plate-girder spans	4.0 cents.
Riveted-truss spans	4.5 "
Pin-connected, Pratt-truss spans	4.5 "
Pin-connected, Petit-truss spans	5.0 "
Cantilever bridges	5.5 "

At the present time, some 6 years later, prices are about ½ cent per lb. less; hence, for this investigation, the writer uses 4.5 cents for fixed spans and 5.0 cents for cantilever bridges. The reason for the greater assumed pound cost of the latter is mainly expensive erection, because cantilevers are generally adopted where the erection conditions are costly.

In carbon-steel bridges the proportion of the total cost of the erected metal which pertains to the erection has been arbitrarily assumed for convenience at 20%, which figure will come very close to the average for any large number of cases.

The cost per linear foot for the tracks has, for convenience, been omitted in computing all curves, as it is constant and common to all spans; but, of course, their weight was included in estimating dead loads.

As no one can foretell what will be the excess costs per pound, delivered at bridge site, for the various alloy-steel bridges over the corresponding costs for carbon-steel bridges, in this investigation

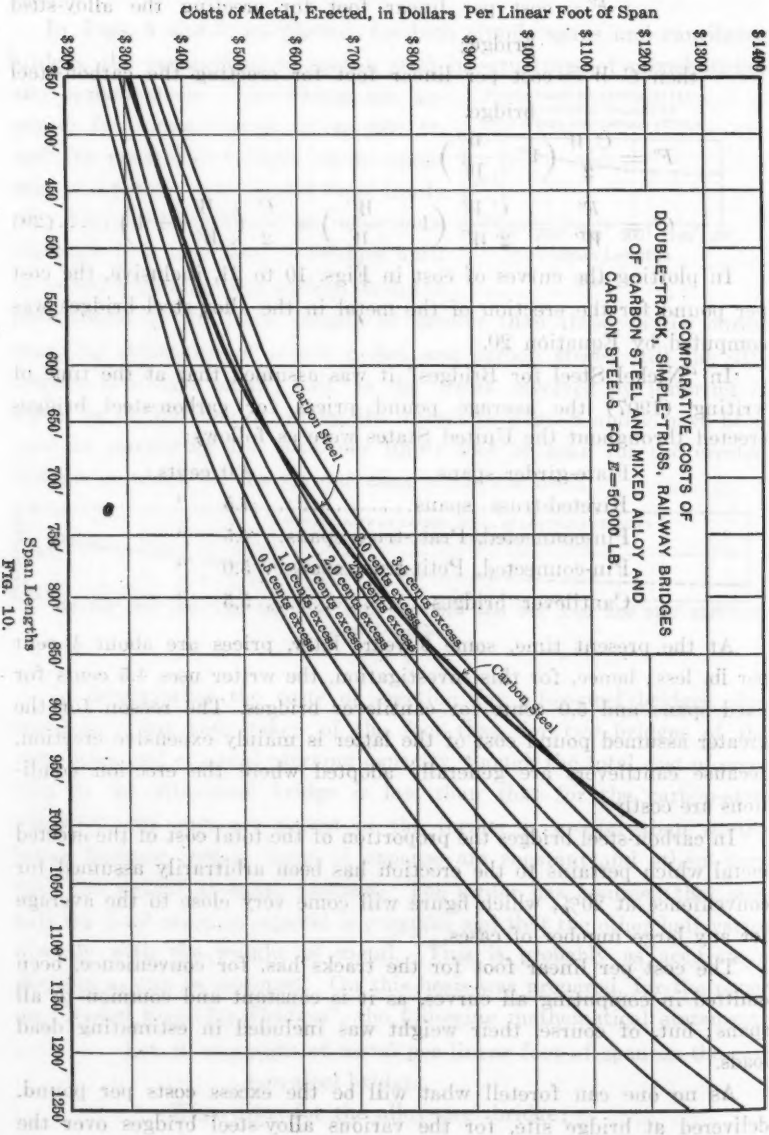

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths
COMPARATIVE COSTS OF DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES OF CARBON STEEL AND MIXED ALLOY AND CARBON STEELS FOR E=50000 LB.
Carbon Steel
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Carbon Steel

FIG. 10.

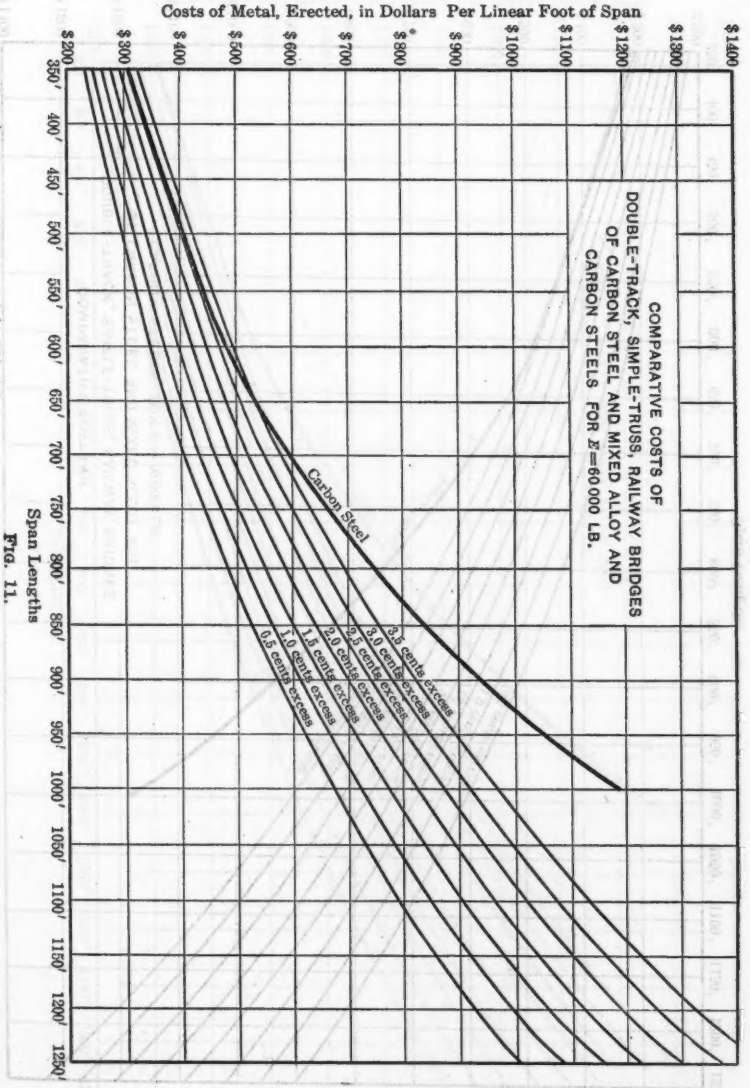

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200 $300 $400 $500 $600 $700 $800 $900 $1000 $1100 $1200 $1300 $1400
350' 400' 450' 500' 550' 600' 650' 700' 750' 800' 850' 900' 950' 1000' 1050' 1100' 1150' 1200' 1250'
Span Lengths
COMPARATIVE COSTS OF DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES OF CARBON STEEL AND MIXED ALLOY AND CARBON STEELS FOR E=60,000 LB.
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

Fig. 11.

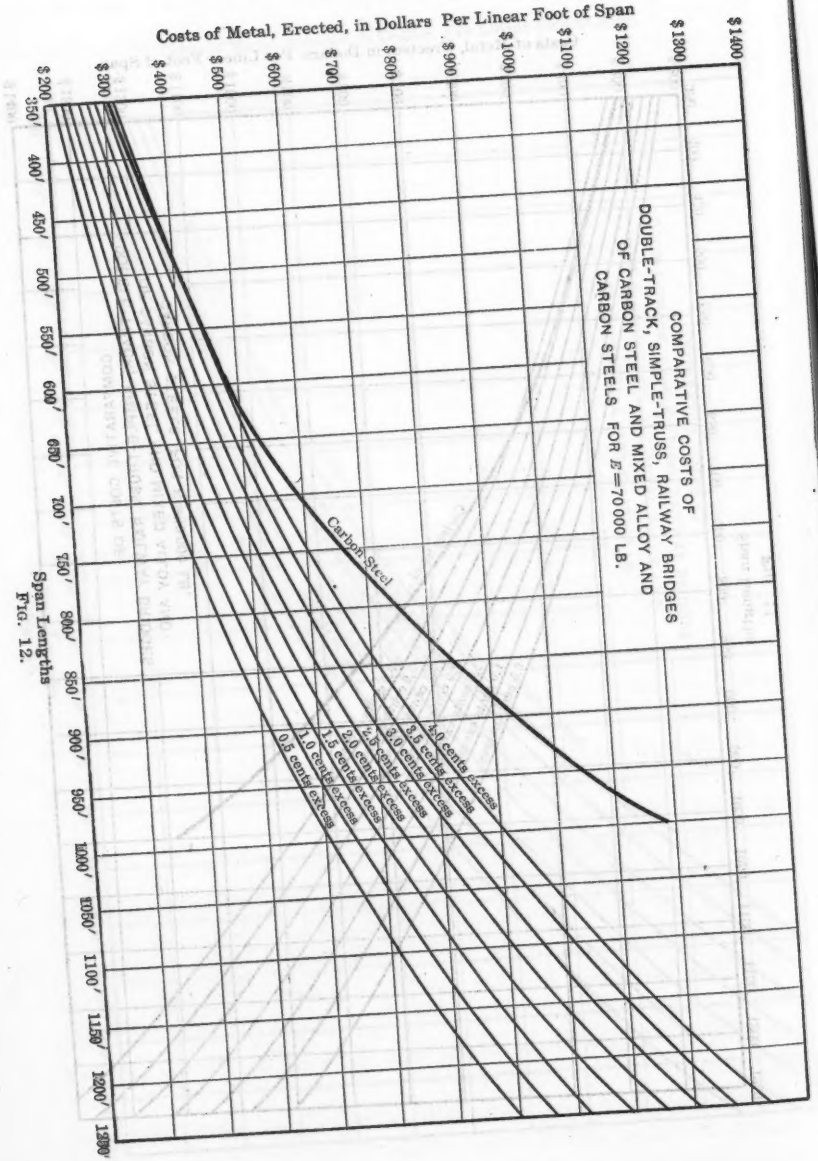

COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=70 000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200 $300 $400 $500 $600 $700 $800 $900 $1000 $1100 $1200 $1300 $1400
Carbon Steel
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350' 400' 450' 500' 550' 600' 650' 700' 750' 800' 850' 900' 950' 1000' 1050' 1100' 1150' 1200' 1250'
Span Lengths

FIG. 12.

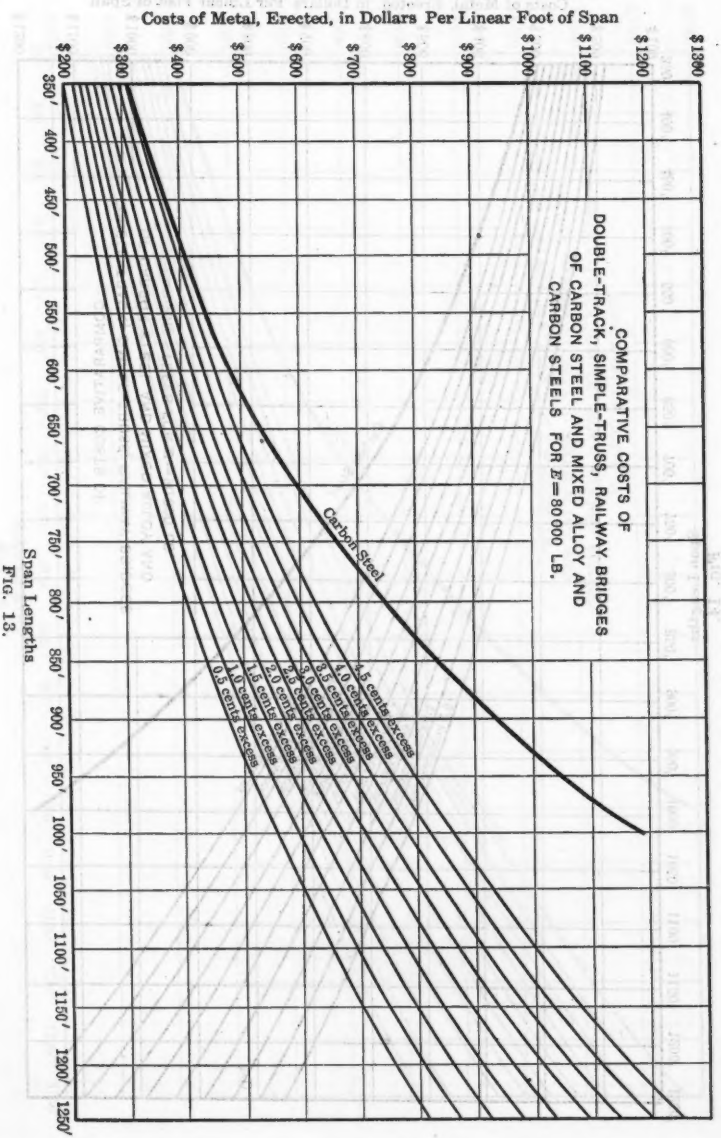

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200 $300 $400 $500 $600 $700 $800 $900 $1000 $1100 $1200 $1300
350′ 400′ 450′ 500′ 550′ 600′ 650′ 700′ 750′ 800′ 850′ 900′ 950′ 1000′ 1050′ 1100′ 1150′ 1200′ 1250′
Span Lengths
COMPARATIVE COSTS OF DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES OF CARBON STEEL AND MIXED ALLOY AND CARBON STEELS FOR E=30000 LB.
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 13.

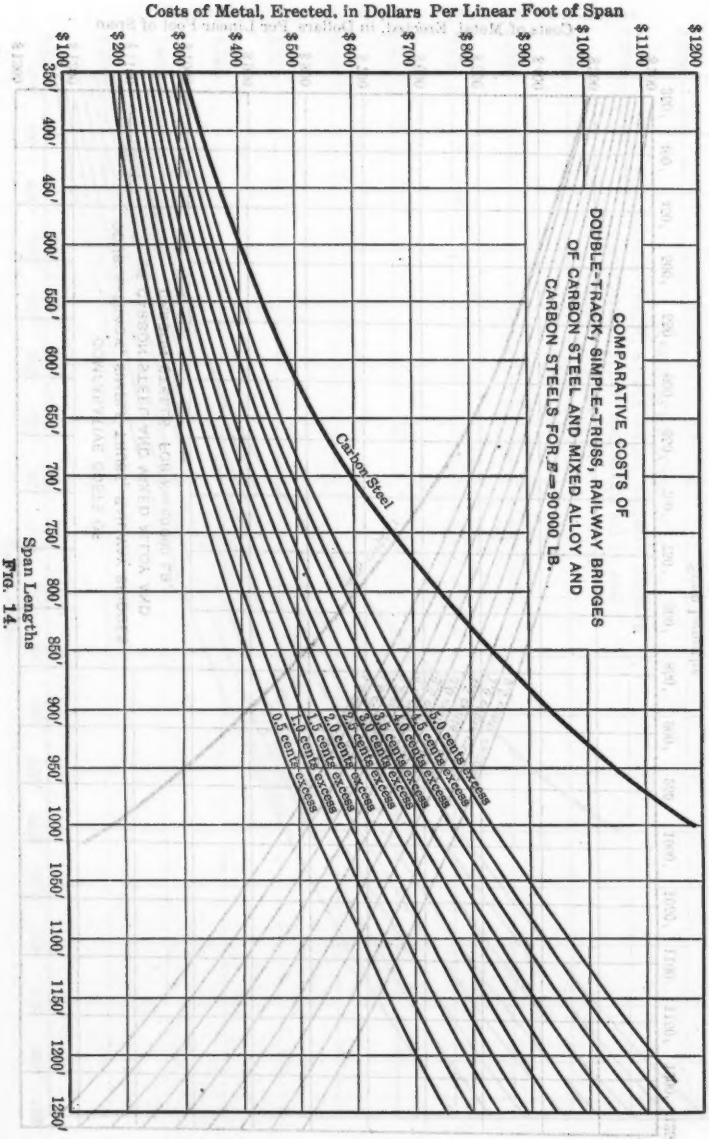

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
350'
400'
450'
500'
550'
600'
650'
700'
750'
800'
850'
900'
950'
1000'
1050'
1100'
1150'
1200'
1250'
Span Lengths
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR f = 90000 LB.
Carbon Steel
5.0 cents excess
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

Fig. 14.

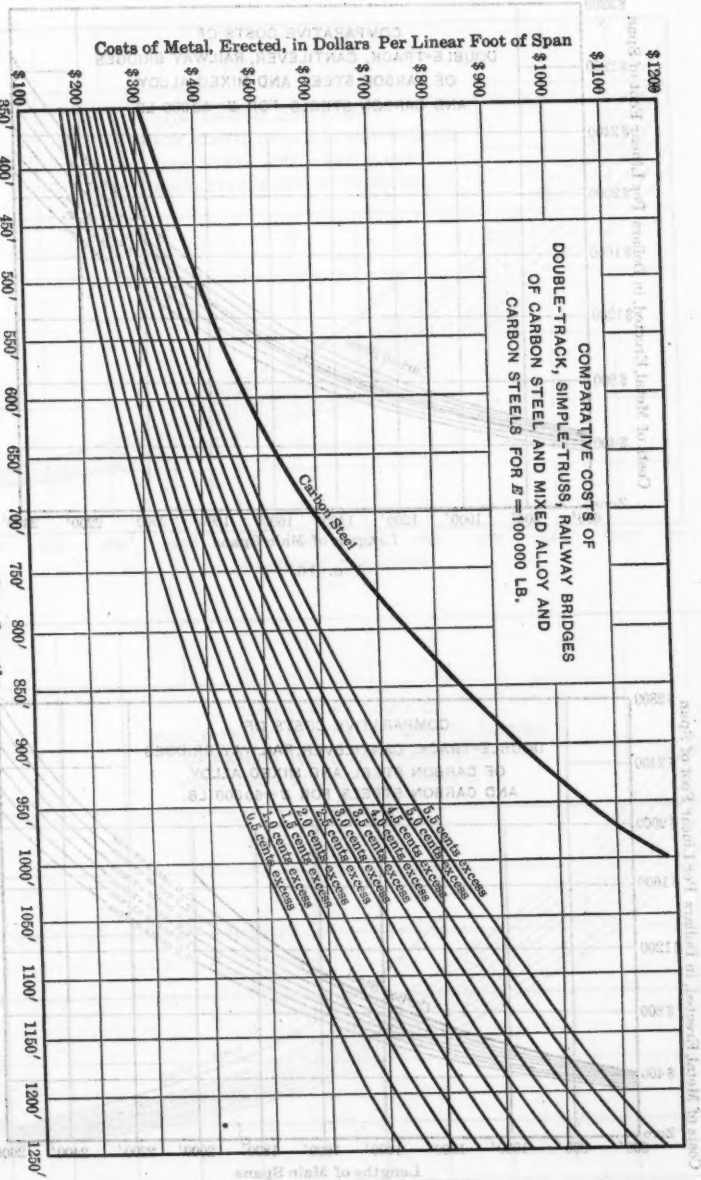

COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E = 100 000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100 $200 $300 $400 $500 $600 $700 $800 $900 $1000 $1100 $1200
350′ 400′ 450′ 500′ 550′ 600′ 650′ 700′ 750′ 800′ 850′ 900′ 950′ 1000′ 1050′ 1100′ 1150′ 1200′ 1250′
Span Lengths
Carbon Steel
5.5 cents excess
5.0 cents excess
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess

FIG. 15.

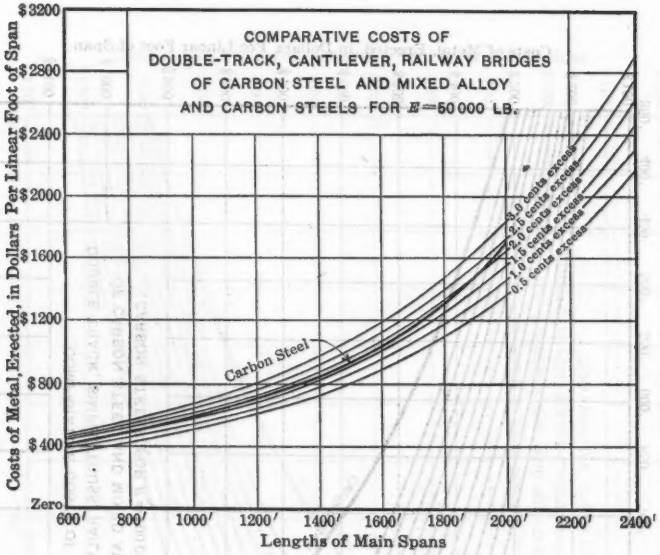

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=50 000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Carbon Steel
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′
Lengths of Main Spans

FIG. 16.

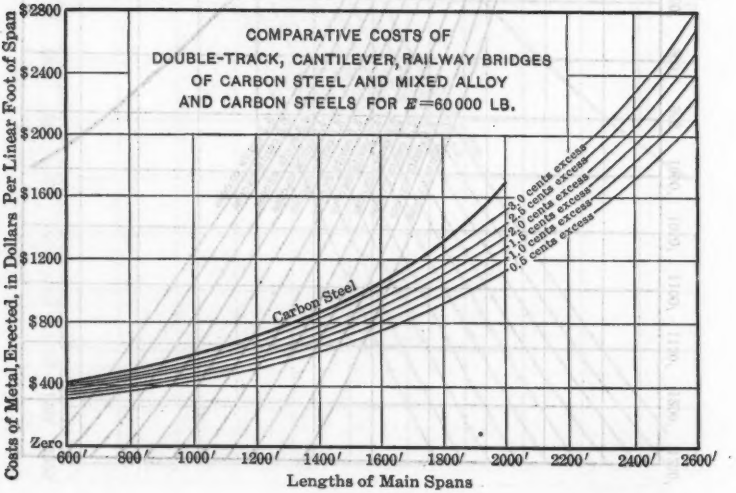

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=60 000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Carbon Steel
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′
Lengths of Main Spans

FIG. 17.

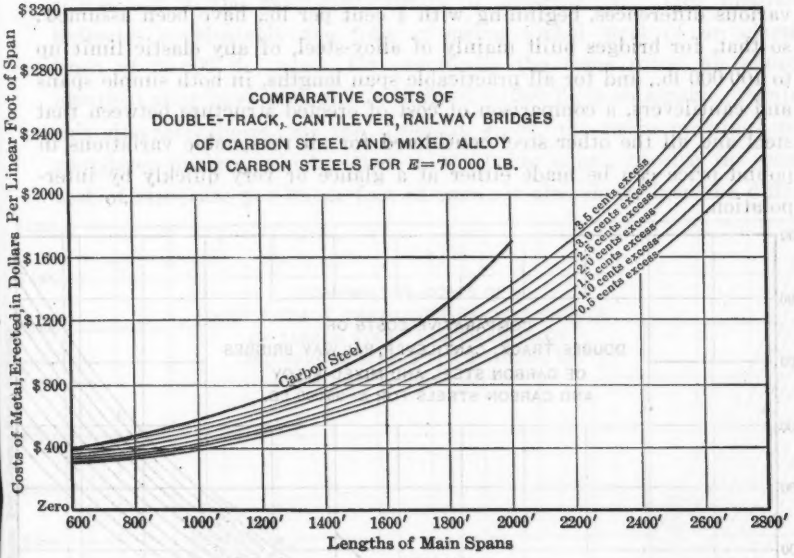

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=70000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′
Lengths of Main Spans

FIG. 18.

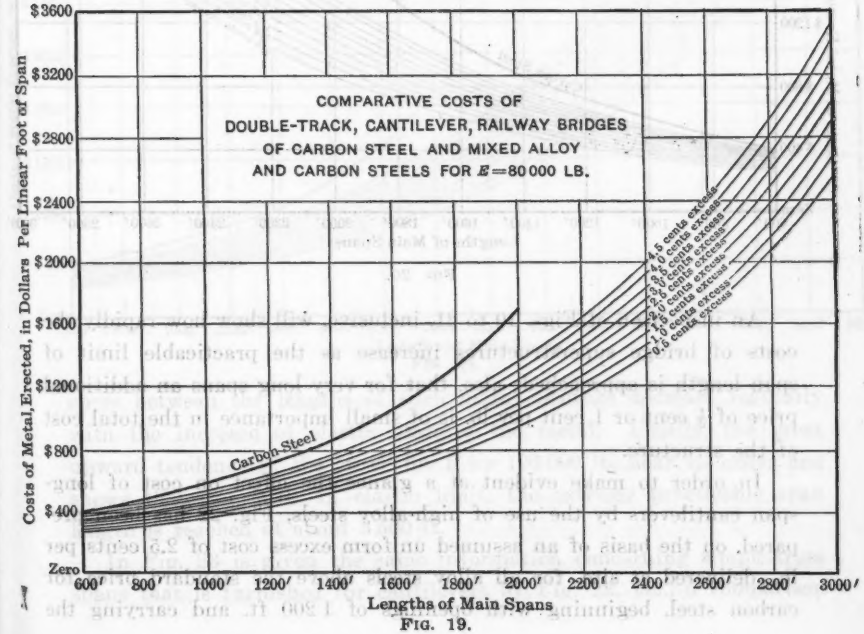

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=80000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3600
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′ 3000′
Lengths of Main Spans

FIG. 19.

various differences, beginning with ½ cent per lb., have been assumed; so that, for bridges built mainly of alloy-steel, of any elastic limit up to 100 000 lb., and for all practicable span lengths, in both simple spans and cantilevers, a comparison of cost of erected structure between that steel and all the other steel considered for all reasonable variations in pound price can be made either at a glance or very quickly by interpolation.

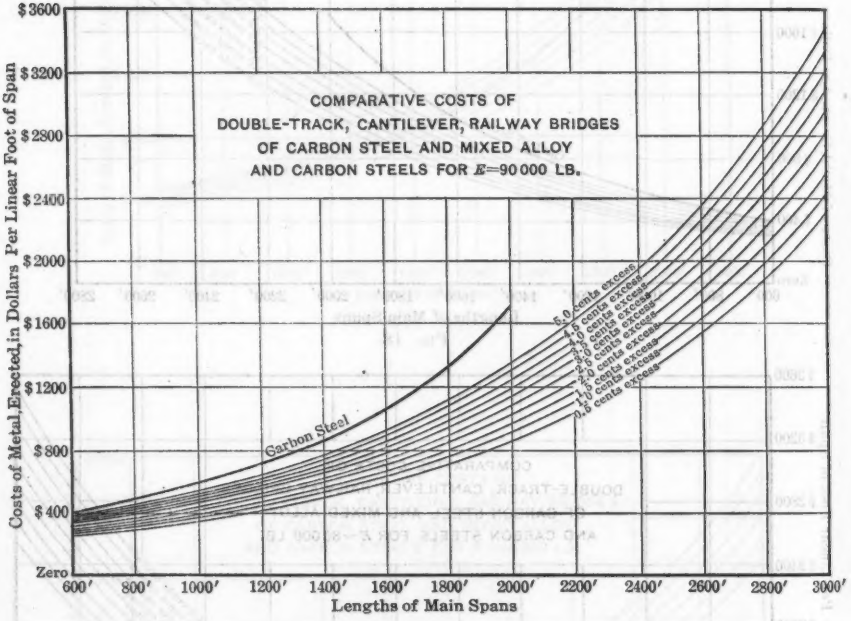


FIG. 20.

An inspection of Figs. 10 to 21, inclusive, will show how rapidly the costs of bridge superstructures increase as the practicable limit of span length is approached; also, that for very long spans an additional price of ½ cent or 1 cent per lb. is of small importance in the total cost of the structure.

In order to make evident at a glance the effect on cost of long-span cantilevers by the use of high-alloy steels, Fig. 22 has been prepared, on the basis of an assumed uniform excess cost of 2.5 cents per lb., delivered at site, for all alloy steels above the standard price for carbon steel, beginning with openings of 1 200 ft. and carrying the

curves out to the practicable limits of construction. This diagram indicates conclusively the folly of using nickel steel of an elastic limit of 50 000 lb., when, for practically the same price, an almost identical alloy having an elastic limit of 60 000 lb. is obtainable. Again, this diagram shows, for the different steels, the main openings for cantilever bridges, which, other things being equal, involve the same cost of structure per linear foot of span; it also shows how the differ-

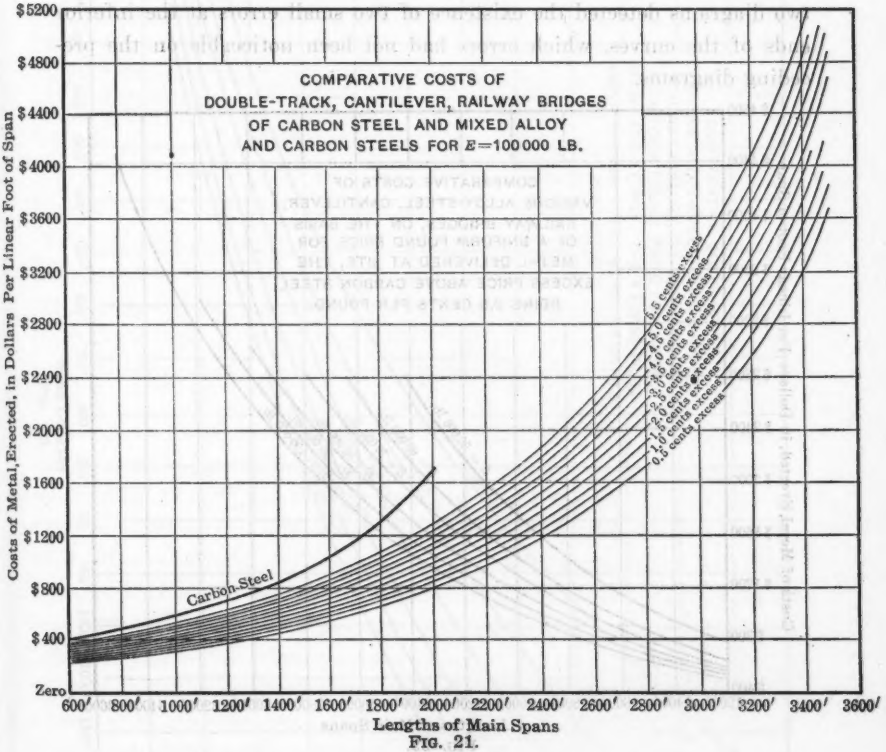


FIG. 21.

ences between the lengths of such main openings decrease regularly with the increase of elastic limit of the metal. Finally, the great upward tendency of the curve for $E = 100\,000$ lb. near its outer end shows that, for such an elastic limit, the extreme practicable span length is reached at about 3 500 ft.

In Fig. 23 is given the same information concerning simple-truss spans that is furnished for cantilevers by Fig. 22, *viz.*, a comparison

of costs of metal per linear foot of span for all the alloy steels, on the basis of adopting a uniform excess price over carbon steel of 2.5 cents per lb.

Incidentally, Figs. 22 and 23 afford an excellent check on the correctness of the writer's numerical computations; for, if there were any ordinary error made, it would be indicated at once by irregularity, either in the curves or their spacing; in fact, the plotting of these two diagrams detected the existence of two small errors at the inferior ends of the curves, which errors had not been noticeable on the preceding diagrams.

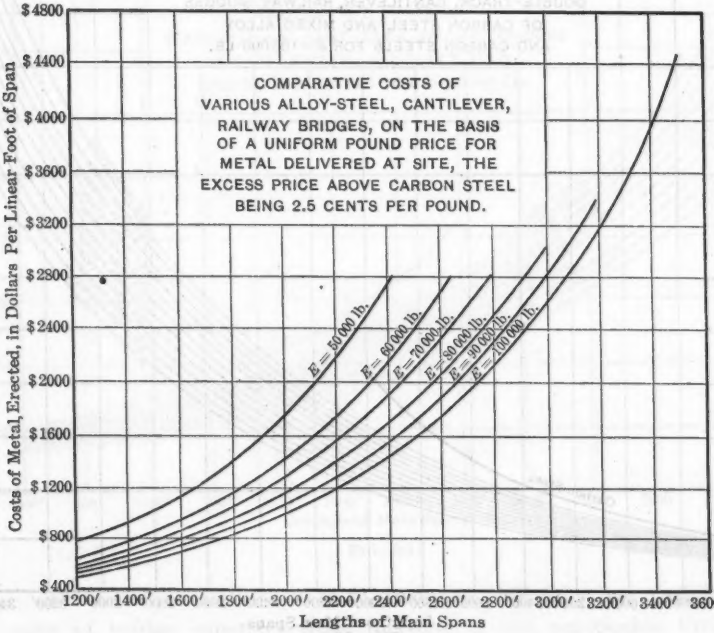


FIG. 22.

This brings up the question of the correctness of all the work in this investigation. There can exist nowhere any small errors of any importance, because they would have been detected at once by the lack of continuity in the diagrams. This feature, though, would not prevent the existence of fundamental errors based on incorrect assumptions or on wrong primary data. Such fundamental errors, however, are really impossible, because the weights of carbon-steel bridges dia-

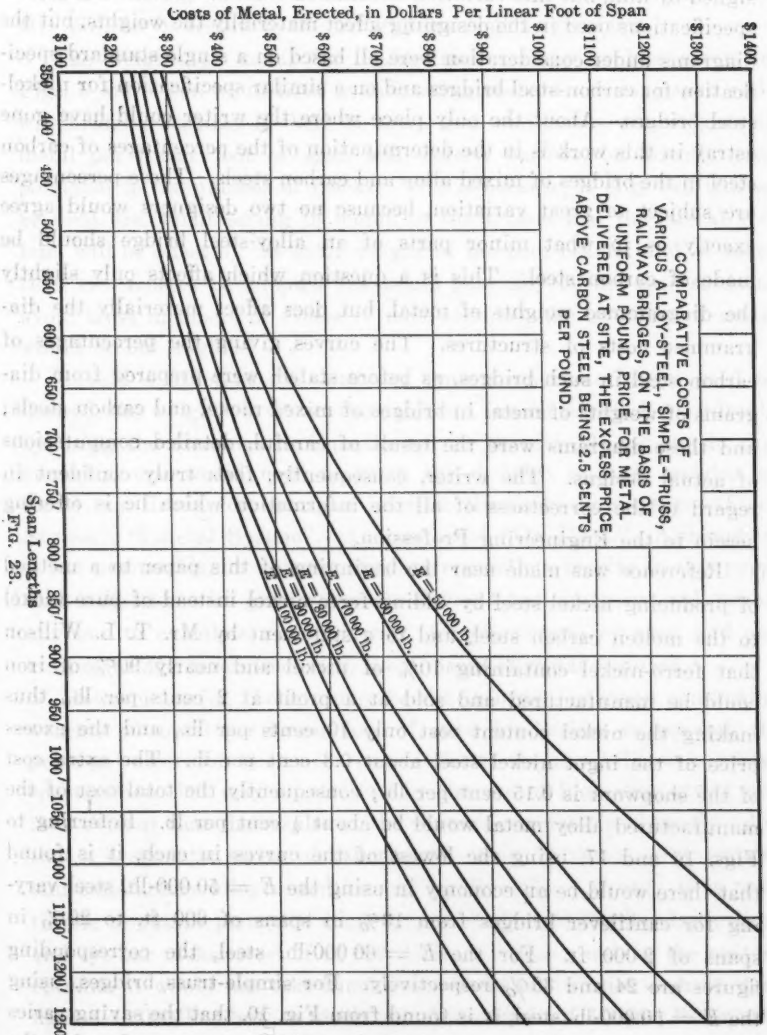

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100 $200 $300 $400 $500 $600 $700 $800 $900 $1000 $1100 $1200 $1300 $1400
350′ 400′ 450′ 500′ 550′ 600′ 650′ 700′ 750′ 800′ 850′ 900′ 950′ 1000′ 1050′ 1100′ 1150′ 1200′ 1250′
Span Lengths
COMPARATIVE COSTS OF VARIOUS ALLOY-STEEL, SIMPLE-TRUSS, RAILWAY BRIDGES, ON THE BASIS OF A UNIFORM POUND PRICE FOR METAL DELIVERED AT SITE, THE EXCESS PRICE ABOVE CARBON STEEL BEING 2.5 CENTS PER POUND.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.

Fig. 23.

grammed in the writer's office are very exact. Of course, the personal equation of the designer affects the weight of metal in structures designed by him, but this variation is confined to small limits. Again, the specifications used in the designing affect materially the weights, but the diagrams under consideration were all based on a single standard specification for carbon-steel bridges and on a similar specification for nickel-steel bridges. About the only place where the writer could have gone astray in this work is in the determination of the percentages of carbon steel in the bridges of mixed alloy and carbon steels. These percentages are subject to great variation, because no two designers would agree exactly as to what minor parts of an alloy-steel bridge should be made of carbon steel. This is a question which affects only slightly the diagrammed weights of metal, but does affect materially the diagrammed costs of structures. The curves giving the percentages of carbon steel in such bridges, as before stated, were prepared from diagrams of weights of metal in bridges of mixed nickel and carbon steels; and these diagrams were the result of careful, detailed computations of actual designs. The writer, consequently, feels truly confident in regard to the correctness of all the information which he is offering herein to the Engineering Profession.

Reference was made near the beginning of this paper to a method of producing nickel steel by adding ferro-nickel instead of pure nickel to the molten carbon steel, and to a statement by Mr. T. L. Willson that ferro-nickel containing 10% of nickel and nearly 90% of iron could be manufactured and sold at a profit at 2 cents per lb., thus making the nickel content cost only 10 cents per lb., and the excess price of the ingot nickel steel about 0.3 cent per lb. The extra cost of the shopwork is 0.15 cent per lb.; consequently the total cost of the manufactured alloy metal would be about ½ cent per lb. Referring to Figs. 16 and 17, using the lowest of the curves in each, it is found that there would be an economy in using the $E = 50\,000$-lb. steel varying for cantilever bridges from 13% in spans of 600 ft. to 20% in spans of 2 000 ft. For the $E = 60\,000$-lb. steel, the corresponding figures are 24 and 33% respectively. For simple-truss bridges, using the $E = 50\,000$-lb. steel, it is found from Fig. 10, that the saving varies from 16% for 350-ft. spans, to 32% for 1 000-ft. spans; and, using the $E = 60\,000$-lb. steel and Fig. 11, the corresponding savings are 26 and 44%, respectively. From the preceding figures of economy it is evi-

dent that it would pay the builders of large bridges to experiment with the use of ferro-nickel in the manufacture of nickel steel for bridge building, so as to determine whether the claims made by Mr. Willson in regard to its use are borne out by the facts.

The weights of metal for double-track railway bridges given in Figs. 5 and 6 can be utilized in estimating costs of any long-span railway bridges, because, if there be more than two tracks, the weights of metal per linear foot of span will, *ceteris paribus,* be directly proportional to the number of tracks. This is because, if only two trusses be used, the small saving in metal, in the trusses and the lateral system, will be offset by the extra weight of the floor-beams; and, if more than two trusees be adopted (as would generally be the case so as to avoid truss members of excessive cross-section), the economy of metal would be but slight.

Should a different live load per track from those herein used be desired, the weight curves on the diagrams can be modified accordingly by using the formulas in Equations 3, 4, 10, and 12; but, to do this correctly, one would need to know the division of total weights of metal per linear foot of span between the four components, "Floor System", "Lateral System", "Trusses", and "On Piers." Although it is not practicable, on account of space restriction, to give in this paper such a division with great accuracy, Tables 1 to 6, inclusive, will enable any one to calculate approximately, for any length of span and any kind of bridge herein included, the division of metal required.

However useful, though, may be the information given concerning weights of metal per linear foot of span for bridges in general, the principal object of this paper is to indicate the possibilities in bridge construction that may be attained by the use of high-alloy steels, and it is evident that the results of the writer's computations clearly prove that a systematic series of experiments made in search of a suitable and satisfactory alloy would be well worth while. Already it is practicable to obtain plate and shape nickel steel of 60 000 lb. elastic limit and eye-bar nickel steel of 65 000 lb. elastic limit; and, in the writer's opinion, it would not take much experimenting to raise each of these figures 10 000 lb.; but, to attain an elastic limit of 100 000 lb. or even 90 000 lb. for an alloy steel suitable for all shop manipulations is truly a great problem, and one worthy of much effort and a large expenditure of time and money.

TABLE 1.—FLOOR SYSTEM FOR SIMPLE SPANS.

Metal mainly used in span.	WEIGHTS OF METAL PER LINEAR FOOT OF SPAN, IN POUNDS.		
	For 350-ft. span.	For 600-ft. span.	For 1 000-ft. span.
Carbon steel	1 400	1 550	2 000
E = 50 000 lb	1 150	1 300	1 750
E = 60 000 "	1 000	1 150	1 600
E = 70 000 "	900	1 050	1 500
E = 80 000 "	850	1 000	1 400
E = 90 000 "	800	950	1 300
E = 100 000 "	750	900	1 200

TABLE 2.—LATERAL SYSTEM FOR SIMPLE SPANS.

Metal mainly used in span.	WEIGHTS OF METAL PER LINEAR FOOT OF SPAN, IN POUNDS.		
	For 350-ft. span.	For 600-ft. span.	For 1 000-ft. span.
Carbon steel	450	600	1 200
E = 50 000 lb	450	600	1 150
E = 60 000 "	450	600	1 100
E = 70 000 "	450	600	1 050
E = 80 000 "	450	600	1 000
E = 90 000 "	450	600	950
E = 100 000 "	450	600	900

TABLE 3.—ON PIERS FOR SIMPLE SPANS.

Metal mainly used in span.	WEIGHTS OF METAL PER LINEAR FOOT OF SPAN, IN POUNDS.	
	For 350-ft. span.	For 1 000-ft. span.
Carbon steel	250	400
E = 50 000 lb	200	300
E = 60 000 "	190	280
E = 70 000 "	180	260
E = 80 000 "	170	240
E = 90 000 "	160	220
E = 100 000 "	150	200

The writer is of the opinion that the first step to take is to experiment on "purified" steel, so as to bring it to its maximum of effectiveness, then to try adding nickel in various quantities, and afterward nickel and other but cheaper substances. Of course, augmenting the quantity of carbon in the purified steel, while increasing both its

TABLE 4.—Floor System for Cantilever Bridges.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800-ft. span.	2 400-ft. span.	3 000-ft. span.	3 600 ft. span.
Carbon steel.....	1 600	1 800	2 000	.....	.....	.....
E = 50 000 lb.....	1 200	1 450	1 650	2 400	.....	.....
E = 60 000 "	1 000	1 200	1 400	2 100	.....	.....
E = 70 000 "	950	1 150	1 300	1 850	.....	.....
E = 80 000 "	900	1 050	1 200	1 700	2 200	.....
E = 90 000 "	850	950	1 100	1 600	2 000	.....
E = 100 000 "	800	900	1 000	1 500	1 900	2 400

TABLE 5.—Lateral System for Cantilever Bridges.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800-ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel.....	800	1 100	1 400	.....	.....	.....
E = 50 000 lb.....	800	1 050	1 300	1 800	.....	.....
E = 60 000 "	800	1 000	1 200	1 600	.....	.....
E = 70 000 "	800	1 000	1 200	1 600	2 000	.....
E = 80 000 "	800	1 000	1 200	1 600	2 000	.....
E = 90 000 "	800	1 000	1 200	1 600	2 000	2 400
E = 100 000 "	800	1 000	1 200	1 600	2 000	2 400

TABLE 6.—On Piers for Cantilever Bridges.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800 ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel.....	700	1 100	2 100	.....	.....	.....
E = 50 000 lb	600	900	1 700	.....	.....	.....
E = 60 000 "	580	860	1 600	2 200	.....	.....
E = 70 000 "	560	820	1 500	2 100	3 500	.....
E = 80 000 "	540	780	1 400	2 000	3 200	.....
E = 90 000 "	520	740	1 300	1 900	2 900	3 900
E = 100 000 "	500	700	1 200	1 800	2 600	3 600

ultimate strength and its elastic limit, would tend to harden the metal; but the addition of nickel (and possibly other elements) would tend to reduce the brittleness and render it workable.

The problem of finding a high, cheap alloy of steel, suitable in every particular for bridges, is now before the metallurgists and the

builders of large metallic structures; and the values of all the results probably attainable are clearly indicated in this paper; hence the onus is on the Engineering Profession to see that the necessary experiments are arranged for and thoroughly carried out, in order that the world may have at its command a new metal that will permit of the spanning of waterways which are so wide and so deep, or are so restricted by navigation requirements, as at present to defy the art of the bridge engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By Messrs. H. P. Gillette, C. B. Burdick, Wynkoop Kiersted, C. W. Hudson, Allen Hazen, and F. W. Green.

Mr. Gillette.

H. P. Gillette, M. Am. Soc. C. E. (by letter).—With very much of the Committee's report the writer is in accord. The most important of its recommendations is that entitled the "Equal-Annual-Payment Method", which involves reduction in the property value, according to the sinking-fund formula for depreciation, and the payment of interest on the residual depreciated value. This recommendation the writer cannot support. He concedes that it is possible to apply the method, under the ideal conditions tacitly assumed by the Committee, but he regards it as impracticable under conditions which actually exist, aside from the likelihood of its being unfair. Moreover, in view of the fact that when interest on sinking fund and on depreciated value are the same, the proposed method gives identically the same rates as would result from the application of a much simpler method, the writer fails to see wherein the complex method suggested by the Committee has any advantage. Its disadvantages are certainly pronounced and many.

The table on page 34 is used to illustrate the application of the Committee's proposed method of "Equal-Annual Payments". Column 2 gives the depreciated value, Column 3 the "depreciation during year", and Column 4 the annual interest on the depreciated value. The sum of Columns 3 and 4 is the total depreciation and interest

*Continued from February, 1914, *Proceedings*.

Mr. Gillette.

for the given year, as shown in Column 6, which is seen to be a constant. Thus, at the age of 10 years, we have:

Depreciation (on $61.96)	$4.93
Interest, 5% on $61.96	3.09
Total	$8.02

This depreciation of $4.93 is the annuity that would amortize the $61.96 in the remaining 10 years of life. This calculation is entirely in accord with a principle laid down by the writer, 4 years ago, in his "Handbook of Cost Data", namely:

"The owner of a second-hand machine is entitled to such a price for it as will enable the purchaser to go on with its use and produce each unit of product at as low a cost as the average unit cost of production would be during the entire life of the machine."

Column 6 of the table on page 34 shows that this condition is fulfilled, and interest and depreciation are a constant amount ($8.02) throughout the life of the plant unit. It follows, therefore, that rates or prices for service would be unchanged, due to depreciation. For this principle of unchanged rates the writer has long contended; but, unfortunately, when a depreciated value has been used as a basis for rate-making, the rate-making body has not recognized the fact that the shorter remaining life of plant units logically necessitates a correspondingly higher depreciation annuity. In other words, rate-making bodies have invariably taken a depreciation annuity based on the full life of plant units, and then, when they have used depreciated values for rate purposes, have applied that annuity to plant units, part of the life of which has expired—a logically indefensible procedure. If depreciated value is to be used as a basis for rate-making, it becomes necessary for engineers to apply correspondingly higher rates of annual depreciation, because of the shortened time during which the owners of a utility are forced to recover their remaining capital. The public, it will be seen, will not gain in lowered rates when this is done; and will then cease to be so eager to assign a depreciated value to utility plants.

If the writer interprets the Committee's attitude correctly, he cannot see that it offers a better method than has hitherto existed for rate-making. The old plan has been to keep the capital intact, always paying interest plus profit (= fair return) on the full investment, and always providing for repairs and renewals out of earnings. In the case given as an illustration, on page 34, this end would be attained by adopting an unchanging depreciation annuity of $3.02 and an unchanging interest of $5.00 on the full investment of $100. Let us consider some of the practical difficulties that would arise were this method abandoned in favor of the Committee's method.

Mr. Gillette.

First, according to the Committee's recommendation, there would be no depreciation fund or replacement reserve; but the utility company would be at liberty to distribute the depreciation annuity among its owners. This carries with it the necessity of calling in stocks or bonds to the amount of the depreciation annuity thus distributed. Conversely, as soon as any renewals of plant units would be made, new securities would be issued. Such a constant flux of securities seems to the writer impracticable for many reasons, which need not be mentioned, as they will be evident on reflection.

Second, the accrued depreciation is entirely a function of the estimated life of each group of plant units, according to the Committee's method. This assumption, however, is not justifiable except for natural depreciation. In the case of accrued functional depreciation (due to obsolescence, inadequacy, etc.), it is impracticable to estimate accrued depreciation by any formula that recognizes only the age of plant units. There is only one accurate method of estimating accrued functional depreciation, namely, by comparison of the unit costs of the product or service rendered, using, as a basis for comparison, the most economic plant unit available. This is a special case of the Unit Cost Formula for Depreciation, which the writer has recently discussed at length elsewhere.*

Even engineers are not always aware that most of the "lives" of plant units given in tables are not natural but functional lives. It is probable that the average functional life of all plant units of utility plants in America has been less than 20 years, whereas the natural lives of many of those same units is so great that as yet no engineer can fix their limit.

Thus, locomotives have had a functional life of about 20 to 25 years in America. In Europe, the functional life has been far greater, due to the slower growth of traffic and to other reasons. What is the natural life of a locomotive? No man knows. Stevenson's second locomotive is still in use in an English colliery, after more than 90 years of continual service.† Engineers have deceived themselves, and have unwittingly deceived others, by publishing tables of the lives of plant units without stating whether it was natural or social forces that limited those lives. The writer has come to have less and less confidence in statements as to lives of plant units when nothing is said as to the conditions and causes that have operated to limit such lives. Yet how rarely is a word given in explanation of those conditions and causes. To say that cast-iron pipe averages 50 years of natural life, for example, is to say what no man can prove; first, because pipe-life data are too meager to be entitled to use in striking an average; second, because short functional life has served so generally

* *Engineering and Contracting*, Oct. 30th, 1912; and *Electrical World*, Nov. 2d, 1912.
† *Engineering and Contracting*, Oct. 11th, 1911.

Mr. Gillette.

to obscure the fact that the natural life is long. Finally, the writer refuses to be bound by averages of any kind as applied to a special case unless it can be shown that the special case itself conforms approximately to the average.

The bearing of all this on the question before us is plain. The Committee suggests a method that involves the calling in of stocks or bonds, and re-issues of the same, depending on some engineers' guesses as to the probable life of many plant units, and, in most cases, the guess relates to functional life.

It will be answered that a guess as to life of plant units must be made in any event, if a depreciation fund is to be established, and that such a guess must involve guessing at functional life. This is true; but there is a marked difference between establishing a depreciation fund that shall be ample, and establishing a depreciated value that shall be fair to an investor whose capital is at stake. In the case of a depreciation fund, an error one way or the other can be rectified. If experience demonstrates, for example, that too large a fund has been accumulated, then depreciation annuities can be reduced. This can happen without injustice to the public or to the public utility company; but, let a mistake be made in calculating depreciated value, let interest distributions be based on an erroneous depreciated value, and a serious injustice may be wrought.

In the writer's opinion, a depreciation fund should be built up especially to provide for functional depreciation; also, to provide for renewals of plant units having a long natural life. For plant units having a short natural life, and where such units are of all ages, such a fund is seldom needed. A depreciation fund is, in essence, a device whereby the original investment is to be preserved intact. As a member of the Pennsylvania Public Service Commission aptly put it: "This is only a method of underwriting the investment."

As previously indicated, the third serious objection to the method proposed by the Committee arises from the necessity of great accuracy in estimating accrued depreciation. This accuracy is not limited to the time of the initial appraisal, but must be perpetual, for the repeated calling in and re-issuing of securities to avoid over-capitalization involves very accurate determination of accrued depreciation when the latter is to be the basis for capitalization. Accuracy in applying a sinking-fund formula for accrued depreciation involves opening a separate depreciation account with every group of similar plant units at the time the group is put into service. It is not correct to use "composite lives" and "composite ages" in calculating depreciation annuities for a sinking fund. Thus, a group of 1 000 poles, 10 years old, cannot be averaged with a similar group 2 years old, and a depreciation annuity calculated for the total life minus the average age of 6 years.

Mr. Gillette.

The fourth objection to the Committee's method is touched on by the Committee itself. A serious difficulty with its "Equal-Annual-Payment Method" arises when the rate of interest used in calculating the depreciation annuity differs from the rate of "interest" (or fair return) on the depreciated value of the plant. Then, as shown in Column 7 of the table on page 34, the "equal-annual payments" cease to be equal, and become progressively less until the plant unit is renewed, when they jump up again. The Committee thinks, however, that current repairs will rise rapidly enough to offset the diminutions shown in Column 7 of the table, but in this it errs. Suppose the plant units having the 20-year life assumed in the table are cedar poles, or any other sort of units not made up of short-lived parts. Then, assuredly, the Committee's assumption of rising repairs fails of fulfillment. Again, let the 20-year life be a functional life (not a natural life), say, of an electric generator or of a railway bridge or station, then long before the age has become great enough to cause much of a rise in the curve of repairs, the entire unit is retired. Under one of these two classes falls most of the investment in plant units of railways and other public utilities. Hence the method advocated by the Committee fails to accomplish the result that it was its aim to accomplish, namely, equal annual payments, and, therefore, stability of rates charged for service. It would seem, then, that the Committee's method results in the very predicament that other engineers have objected to, namely, variable rates for service if depreciated value is the basis for rate-making. The Committee's criticisms, therefore (see pages 48 and 49 of its report) of the views of other engineers on this matter are not entirely answered when it presents a method which is itself subject to the same criticism.

Apparently, the Committee's main objection to using the new value of a plant as one of the elements in rate-making, arises from the belief that by so doing the public will be made to pay twice for the accrued depreciation. The writer's experience thus far has been that the public has rarely or ever recouped the railways and other utility companies for the accrued depreciation. Depreciation funds seldom exist, and, where they do, they still more seldom equal the accrued depreciation. What is more significant, however, is the almost universal development cost, or accumulated deficit in fair return, that railways and other utilities disclose when their accounts are properly analyzed. When, in calculating development cost (= the "going value" of the Wisconsin Railroad Commission), a depreciation annuity is allowed as a part of the past operating expenses, sufficient to provide for the accrued depreciation shown by an appraisal, it is a rare occurrence when the development cost does not more than equal the accrued depreciation. This is one of the surprises that a much deceived public is to receive. Muck-raking articles based on the flimsiest of evidence,

Mr. Gillette.

grossly distorted, have created a very general impression that nearly all public utility stock is badly watered, and that enormous profits have been universal. Engineers, who should know better, are not entirely free from a bias created by this false "testimony".

Accrued depreciation receives its proper consideration in calculating development cost, or the cost of establishing the business. The writer has discussed this matter at some length elsewhere* and, therefore, will not go further now than to caution against a blind adoption *in toto* of the Agency Theory on which the development-cost calculation rests. Although it is wise to determine what would be the total investment (physical and non-physical) according to the Agency Theory, it should not be forgotten that the theory has evolved slowly and is comparatively new. To what degree a new theory can rightly be made retroactive is a matter for serious consideration.

These and other difficulties confront rate-regulating bodies which aim to be fair. As yet there seems to be no escape from the necessity of weighing all the facts bearing on value as well as on cost, as outlined in Justice Harlan's celebrated decision (1898) in the Smythe *v.* Ames case.

The Committee's report indicates an effort to compromise between the Agency Theory and the Replacement Theory, by taking past conditions and present prices. Is it not better not to mix the theories, but to present the results of each theory complete and logically derived? Then the final "rate-making value", which is likely to be a compromise, will be arrived at after weighing all the evidence. The engineering mind usually rebels at a compromise of any sort, but in appraisal cases compromises are almost inevitable, primarily, because we are in a transitional period. The old Competitive Theory is rapidly being modified by the new Agency Theory of rate regulation. Not only does each theory have its merits, but it is impracticable to make the new theory entirely retroactive without working grave injustice in many instances.

Mr. Burdick.

C. B. BURDICK, M. AM. SOC. C. E. (by letter).—This report is particularly opportune, for we have just entered an epoch of rate regulation; and the valuation of the railroads lately undertaken by the Interstate Commerce Commission has directed many able minds to the subject for the first time. It is not surprising that, under these circumstances, many ideas strongly advocated are very discordant with one another. To the casual observer it might seem that the principles of valuation are in a very chaotic state, or, as the Committee expresses it, that they are in a developmental stage. The principles involved, however, are much more clearly established at this time than appears on the surface.

* *Engineering and Contracting*, June 26th, 1912.

Mr. Burdick.

The Committee has apparently attempted to formulate a procedure for fixing an equitable value that may be used in the matter of rates, by considering each item of value by itself, as to its equity or inequity, on the assumption that, if the parts are equitable, the sum of the parts must be. The defect in this procedure lies in the impossibility of determining what is equitable in most cases until after all the parts have been summed up and considered in the light of all facts having a bearing on value. When the sum is obtained, it represents nothing in particular. It is neither present cost of duplication, nor investment, and probably very imperfectly represents value.

It will greatly clarify the discussion to decide at the outset whether the fair return to be established shall apply on investment or on the value of the property.

If public utilities were not yet established, it would seem entirely equitable to all concerned to provide that the rates should be based on a fair return on reasonably wise investment. The public could have no cause for complaint, and the investor, being reasonably sure that his investment would be protected, would probably be content with a low rate of return. Eras of high and low prices would affect injuriously neither the investor nor the public. Those looking for large profits would invest elsewhere. However, no such situation exists. In practically all cases at this time we are dealing with established properties, nearly all of which antedate the epoch of rate regulation and the Court decisions based on public policy that have made rate regulation possible. The Constitution of the United States provides that private property shall not be taken for public use without just compensation.

The Courts, which must be our guide in the interpretation of the law, are very firm that rate regulation shall be based on the value of the property concerned and, therefore, in valuations for rate-making purposes, it is the duty of the appraiser to determine value as nearly as can be done under the circumstances existing at the time of the inquiry.

How, then, shall we measure this value? The Courts are authority for the belief that cost of duplication under present circumstances is an important measure; that the actual investment should be considered; that even the stocks and bonds and the revenues derived from operation are not without the pale of consideration; that all these should be given their proper weight, under all circumstances, in throwing light on the value. Further, that the value must be reasonable to the buyer and seller, and, in the case of rates, that they must be equitable under all circumstances to the owner and the public.

In various cases involving property the Courts have held that the owner is entitled to the accretions in its value from whatever causes they have arisen, and if the property has depreciated that must be taken into consideration, if for any reason the property must be

Mr. Burdick.

valued. The owner must receive the benefit and suffer the loss of value. In taking his property for the public use he must be remunerated for the property as it exists at the time taken. In fixing values for the purpose of rates and values for the transfer of ownership, it would seem that value in the two cases is one and the same thing, the fair present value of the property; and, so far as observed, the Courts have made no important distinction in the values for these two purposes.

For the same reasons of public policy that permit rate regulation, it is to the public interest that wise investment should be reasonably protected. The property must be allowed to earn rates that will enable it to be self-supporting. It is in the interest of public policy. Unless the property may earn such rates, the service cannot exist. Therefore the Courts have expressed an interest in the amount that has been invested and are loth to fix values, or rates of return, or both, which would tend to discourage investment in like enterprises. For the same reason inquiry as to the stocks and bonds is pertinent, and though the individual security holders cannot be protected against the operations of frenzied finance, the reasonableness of the financial operations can be determined in the light of the figures on investment and cost of reproduction. What light would be thrown on the matter by an estimate of the cost of reproducing the property at present prices under the "conditions existing at the time the various portions of the property were built?" It would throw light neither on the investment nor on the present reproduction cost of the property.

It would seem that the Committee has shown a leaning toward investment as a measure of value for purposes of rate-making, but through the influence of Court decisions, which are numerous and strong, had been forced to the adoption of present prices in the estimates of value. This, it is believed, has placed it in the indefensible position (Conclusion 7) of reproducing the property under the "conditions existing at the time the various portions of the property were built, but on the prices prevailing at or near the time of the valuation." This amounts to assuming part of the circumstances in the past and part in the present, for unit cost is as much a circumstance of the time as any other element subject to change with time. In justification of this conclusion, a case was cited of a reservoir in the construction of which a railroad track was necessarily moved to a new location, the cost of removing the track being properly included in the value of the reservoir. The conclusion is further applied to exclude pavements laid over water pipe after the pipe is laid, and to include the extra cost of "piecemeal construction" in computing the value of water mains. It does not require this general conclusion to justify the inclusion of the railroad track. If we reproduce under the circumstances of the present, we are bound to assume reasonable conditions,

Mr Burdick.

and if a railroad formerly occupied the site of the reservoir, it would be entirely reasonable to assume that it would necessarily be removed in reproducing at the present time, unless there should be good reason to assume otherwise. In reproducing such a reservoir, it would be necessary to assume the contour of the ground, and one could be guided in such assumption only by approximating conditions known to have existed in the past. In this case the past is a guide to reasonable assumptions for duplicating under present circumstances. The cases of pavements and "piecemeal construction", the exclusion of the former and the inclusion of the latter, in the sum total of value, are reversions to past cost or investment, and would seem to have no place in present cost of duplication or present value.

It would seem most logical in the valuation of these properties to pursue one logical method through to the end. If it be the cost of duplication at present, then reproduce under present circumstances, and when the figures are summed up, one will have a total that represents something tangible—the cost of duplication. If it be past cost, an estimate of the investment, then follow past cost consistently to the end, and the total will represent investment. Neither total will necessarily represent value, but the two will point strongly toward it, particularly if they are not far apart.

The Courts state that the rates produced must be equitable both to the owner and to the public, and it is here that an opportunity is offered to test the reasonableness of the totals previously determined. If fair rates of return applied to the total cost of reproducing the entire property necessitate rates that are reasonable to the public, then all the equities are satisfied. If the rates necessitated are unreasonable, then the value or rate of return must be reduced to such amount that the inequity is distributed in a just way between the owner and the public.

In some cases the foregoing procedure is difficult, but in most cases an equitable decision is the result of industry and intelligence in marshaling all the obtainable facts and common sense in their application to the problem particularly in hand. It is not a task for a novice. It requires broad experience in every important phase of the property to be valued, an open mind, and some considerable acquaintance with the subject of valuation in general. A realization of the truth stated by the Committee, that rates too low are unjust to the public, is helpful in establishing justice.

The Committee frankly states, after pointing out the necessity of following the Courts:

"And while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

Mr. Burdick.

Is this a wise position? Has the Committee had the opportunity to consider the matter sufficiently to warrant it in recommending procedure at variance with the law as established by the higher Courts? The law is the outgrowth of the experience of centuries. It is necessarily modified from time to time as new conditions arise. A great concession in favor of the public was made when rate regulation was permitted in the face of existing contracts, but the Courts stand firm as a rock against some of the views advocated by the Committee.

Mr. Kiersted.

WYNKOOP KIERSTED, M. AM. SOC. C. E. (by letter).—The writer has been very much interested in studying this report, which is certainly instructive; but, before the Committee makes its final report, it is hoped that the application to a composite property of the Equal-Annual-Payment Method as well as of the other recognized methods of computing depreciation will have been shown.

The question of rate-making requires the ascertainment, among other things, of a fair earning value of a public utility and a rate of return thereon which is fair to the investor and to the public, and which will provide a fund for replacements and renewals to an extent which, when invested in replacements, will maintain the property in an efficient condition and will protect the original investment. The decision of the United States Supreme Court in the Knoxville case appears to confine the fund for depreciation to replacement of units to an extent which will maintain the original investment unimpaired. Such replacements are expected to be made periodically as needed to keep the property in an efficient operating condition, and when so made it is difficult to escape the conviction that a property thus maintained should be entitled to a return on substantially a 100% value. The writer expressed this view a little more in detail in his discussion of the paper* by John W. Alvord, M. Am. Soc. C. E., on "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return". To what was said in that discussion may be added the suggestion that the earning value of a composite property, made up of the earning value of its units, is not affected by consideration of age as long as repairs of the unit and replacements of parts of the composite property receive timely attention. A unit, retained as a necessary part of an efficient property, possesses a 100% earning value, whether old or new, quite as much as the composite property itself. The question of accrued depreciation, as applied to either the value of the unit or the value of the composite property, does not arise for consideration in a rate case as it does in a valuation for purchase and sale.

The life of a unit of a composite property depends on such a variety of conditions as to be indeterminate with precision. The best evidence in this regard is that furnished by a study of the cost of

* *Proceedings*, Am. Soc. C. E., February, 1914.

operation, maintenance, and replacement of properties which have been in use long enough to afford a wide range of operating conditions. Such information collected from each class of property, properly analyzed, should be the best guide as to the proper rate of return on an investment to cover the cost of replacements. Doubtless a large amount of information of this character has been accumulated through various sources, which should aid materially in the solution of such problems. Mr. Kiersted.

Although the rate of return as interest or dividends, and as profits, may be practically uniform for some classes of property in particular districts, it does not follow that the rate of return to supply a fund to take care of replacements shall be correspondingly uniform. For instance, a public utility in a rapidly growing city requires proportionately heavier expenditures in replacements during the earlier than during the later periods of its existence, whereas the reverse may be true of a slowly developing utility in a slowly growing city. The rate of return for replacements in one case should evidently be more liberal than in the other. Moreover, many physical conditions and service requirements will have a modifying influence to an extent which requires a close analysis of the individual property.

Any method dealing with a rate of return on a public utility, considered as a whole, must be based finally on equal annual payments, and will be divided essentially into two parts, namely, a rate of return to the investor as interest or dividends, and as profits, and a rate of return to cover replacements. As long as the rate of return is equitable for both purposes, it does not matter what the detailed analytical methods may have been in reaching a conclusion, if that conclusion is substantially correct. In order to reduce the probabilities of error of individual judgment as much as possible, it were perhaps as well that the analytical methods, taking into consideration the units of a composite property, should embrace the use of the several rules or mathematical formulas rather than that of only one rule or formula.

The valuation of portions of a physical property on the basis of present value under original conditions, and other portions on the basis of present value under present conditions, introduces an uncertainty as to whether such a distinction should be made, and, if made, whether the reasons therefor are sufficient to justify the distinction. Furthermore, the unearned increment included in an item of property, particularly real estate valued under present conditions, raises the question as to the extent which this unearned increment of value is to be considered in a rate case, and whether, if considered, it should earn a rate of return equal to that of other portions of the physical property, barring a return for replacements. Still further consideration is open to the question of whether such an item of property as real estate, possessing value independent of the public utility with which it may be associated, by reason of urban surroundings and asso-

Mr. Kiersted.

ciations, is subject to the same degree of hazard of ownership as that attaching to other items of property possessing practically no value except as a part of the public utility. If the hazard in the one case is less than in the other, should the owner be entitled to as great a return on the portion of his investment involving little or no hazard as on that portion involving a positive hazard?

Mr. Hudson.

C. W. HUDSON, M. AM. SOC. C. E. (by letter).—It appears to the writer that the Committee should have confined its report to the field for which it was appointed. Undoubtedly it will be conceded that engineers have special knowledge on the subject of cost of reproduction of the physical elements of public utility property. Assuming that the Committee has been able to formulate principles and methods which will be of use in determining the value of the property of railway and public utility corporations, it by no means follows that it has been or that it will be able to show how the values thus determined may be used to fix rates.

In the case of a public utility engaged in supplying one commodity to a limited district, the cost of the service may bear quite a close relation to the value of the plant, and in so far as that value affects the reasonableness of rates for service, it might be taken as the cost of reproducing new, the cost of reproducing new less depreciation, or the amount of capital invested. For the case of railway property, which is used under such varying conditions and, in the aggregate, represents nearly one-sixth of the wealth of the nation, it will not be possible to establish to the satisfaction of a majority of the members of this Society a working relation between value and rates.

The making of the tariffs of the railroads is in itself such an intricate and extensive subject, involving as it does the stability of our entire economic structure, that the writer does not believe the Committee was warranted in trying to formulate principles of valuation for rate-making. It is certain that a careful study of the report does not show how it would be of the slightest service in the preparation of proper railway tariffs.

Numerous reasons may be cited to show that even the cost of the service, which includes the value of the plant, should not always determine the reasonableness of a rate. An important requisite is that the traffic must move; that is, that the rate for the service shall be low enough to allow it to move.

The Committee does not appear to include the most valuable elements in its valuation for rate-making. To quote from the report:

"Justice and equity require that in any regulation of rates fixing the amount which may be earned by a public service corporation there shall be taken into consideration two distinct features:

"First, the annual return covering interest and profit, to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment. Mr. Hudson.

"Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

In other words, the Committee would be willing to insure the railways an attractive earning capacity based on these two considerations. To the writer's mind, the owners of railway property could lie down and rest in perfect repose as far as any fear as to what their competitors were doing was concerned. Every business enterprise in order to succeed must have brains, honest labor, and loyalty back of it. To base a rate for service on any consideration of value of the physical plant alone, leaving out the more important factors, would be a great economic mistake, and it will be a serious error for the United States to do any such thing.

It also appears advisable to the writer to separate the rules and methods for valuing public utility properties from those for valuing railway properties.

He does not believe it will be possible for the members of this Society to come to anything like a general agreement as to how railway rates are to be fixed. Very few of them have an extensive knowledge of the subject, nevertheless, there are probably many members who are willing to take it out of the hands of experts. The writer wishes to express his conviction that the Society should not publish any rules for valuation for the purpose of rate-making.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—The writer has the feeling that under the heading, "Present or Original Conditions", the Committee has given more attention to the history of the plant than is fairly justifiable. For instance, in estimating the cost of reproduction of a water-works plant, the cost of cutting through the paving of the street over it, and replacing such pavement after laying the pipe, is unquestionably a part of the cost of reproduction, and must be included in any true estimate of it. The Committee, however, would exclude all that part of the cost which relates to paving laid after the pipe was placed. The writer is unable to see that the question of whether the pipe or the paving was laid first has anything to do with the cost of the reproduction of the property as it now stands, or with its value. It would cost just as much to replace the pipe in one case as in the other. The value of the property, as measured by its capacity for useful service, is identical in both cases. Mr. Hazen.

This procedure, together with others in the report, seems to have been adopted by the Committee in an endeavor to get at something

Mr. Hazen.

that may be taken as the equivalent of "invested capital" to be used in place of the actual amount of the investment, which may be unknown. The impression is given that the Committee had in mind something like the following: What amount of capital might reasonably have been invested in the enterprise, assuming that the prices of the last few years had obtained during its entire history? This idea seems to have displaced to a considerable extent that of the cost of reproduction.

Going a little further, the distinction between cost and value does not seem to be made, and the whole report seems to be directed to consideration of cost. So far as the writer can find, value is not considered in it at all. It may be that in a subsequent report the relations of cost to value will be discussed. Otherwise, it would seem better that the title of the report should be changed, and that cost, or cost of reproduction, should replace the word "valuation" in it.

Mr. Green.

F. W. GREEN, ASSOC. AM. SOC. C. E. (by letter).—This report is a notable addition to the literature on one of the momentous matters of the day, and the comprehensive, and withal temperate, treatment by the Committee of so difficult a subject is, in every way, praiseworthy. A thorough discussion undoubtedly will assure a more perfect understanding of the matter; and a clear understanding must precede a perfect judgment.

The attitude of a very considerable number of persons toward the railways as a whole is suspicious, if not hostile; it seems to have been reached by a most unusual method of logic: (1) *A, B,* ..... *Z* are railway companies; (2) *A* and *J* have been corrupt, inefficient, and arrogant; Conclusion: *A, B,* ..... *Z* have been corrupt, etc. This is a general conclusion from a specific premise.

Again, assume two roads, *X* and *Y*. Road *X* was honorably conceived, constructed, and operated. It replaced its equipment and maintained its plant from adequate reserves from earnings, and it was not necessary for it to capitalize such replacements and maintenance. Road *Y* pursued the opposite policy. Should *X* be condemned for the sins of *Y*?

Two issues present themselves in connection with the consideration of the unearned increment in land values. If other land owners are permitted to enjoy this increment, may railways justly be deprived of it? If railways are not to have equal benefit of the laws, why does the State assess taxes on the basis of the increased value? This brings up the matter of the railway charter. In the usual form, it is in the nature of a contract between the State and the corporation. On the part of the State there is the obligation to do certain things, namely, the grant of authority to buy and sell, to contract, to exercise the right of eminent domain, to locate, construct, and operate a railway, to sue and be sued, etc.; and, on the part of the corporation, in return for

Mr. Green.

these grants, it agrees to do certain things, namely, to furnish adequate transportation service, at reasonable rates, and without discrimination; and this implies the development of new country, the increase of private and public wealth flowing therefrom, and the foundation for a large unearned increment for those land owners who may be fortunate enough to be located near the new development. Some writers aver that the increment in railway right-of-way values is the result of the community development; but if this is so, what caused the community development in the first place?

As to the various Court decisions, it is the writer's belief that in some instances these have erred, either from a failure to present the issue to the Court in terms of the Court's comprehension, or from failure of the evidence to clarify and illuminate the issue with sufficient detail. It is no uncommon experience for engineers to have great difficulty in communicating a more or less technical matter to eminent lawyers in such a way that it will be clearly understood. Again, there appear to have been some cases, at least, in which the issue was confined to the consideration of the strictly legal aspects of the case, rather than to the matter in its economic aspects.

The matter of efficiency of operation and special advantages of location is of absorbing interest. If the well-located line is denied a reward for efficient management, there will be no incentive for it, and the public will be required to contribute to the extravagance resulting. Again, if the rates are to be measured by the amount required to sustain the strongest line, what will become of the weakest? If the rates are adapted to the necessities of the weak line, will not the strong line enjoy inordinate profits; or, if a fat rate is permitted on the weak line, and a lean rate on the strong line, will not development move along the line of least resistance, and population and traffic increase on the strong line and decrease on the weak line? If so, will there not be a discrimination established between communities, which will produce much more public discontent than under the present alleged unsatisfactory conditions? It is along these lines that the real difficulties of applying the theory of making the rate a function of the valuation become apparent. The Committee has very wisely remarked that other considerations should enter into the subject of rate-making. It may be pertinent to mention some of them: The competition of carriers; the competition of commodities; competition of communities; value of the service to the patron; cost of performing the service; density of traffic; direction of traffic; volume of traffic; carload minima and maxima; slow or fast movement required; and insurance as carriers.

As to depreciation: Some hold that rates should be based on the depreciated value of the property, unless and except, the depreciation fund in cash is kept on hand intact, and so denominated. The usual

Mr. Green.

argument is that, if such fund is not thus kept and labeled, there is a return of capital to the owner of the utility, and hence the capital invested in the first instance is withdrawn and available for reinvestment. It is to the user's interest that the property be maintained, both with regard to adequacy and economy. As long as efficient service can be given, the user should expect to pay to the owner as much for the same service on an old road as on a new one; and as there is no requirement that maintenance be paid for out of any other than the general funds of the owner, and further, because adequate maintenance is, or should be, a prior lien on such funds, there would seem to be no justice in making the rate fluctuate with the variations of physical condition. Moreover, if it is conceded that there is in effect a return of capital to the owner, does not this carry with it the implication that it would be permissible for the owner to capitalize maintenance charges when made, and would this not be in violation of the principles of accounting established by the Interstate Commerce Commission?

It is to be hoped that in due time will come recognition of the economic truth that the welfare of all depends on the just and equitable treatment of each unit in the body politic.

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PAPERS AND DISCUSSIONS

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ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion.*

By Messrs. Nelson P. Lewis, John C. Trautwine, Jr., William Goldsmith, George A. Ricker, George W. Tillson, Robert A. Meeker, R. E. Beaty, William M. Kinney, D. B. Goodsell, H. W. Durham, Herbert Spencer, T. Hugh Boorman, and Philip P. Sharples.

ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

Nelson P. Lewis, M. Am. Soc. C. E. (by letter).—It being admitted that an engineering organization is required to deal with highway work, it follows that the details of such an organization should be determined by engineers. The time has passed when engineers should be expected to carry out municipal work of any kind when they are told, as they are told, and because they are told. Municipal engineers must assume the responsibility of determining, not only the kind of organization through which this control is exercised, but the general policies of a city with respect to such work. Every city needs a definite plan and programme. The need of a comprehensive plan for a street system is now generally recognized, but it is not as generally recognized that this should be the work of the engineer. A paving programme is just as essential. The policy of most American cities has been to leave the kind of pavement to be laid on any street to be determined by the abutting property owners, on the ground that, as they are to pay the bills, they are entitled to say what kind of pavement they should have. Such a plan cannot result in a well-paved city. By a well-paved city is meant a consistently paved city, that is, a system of pavements in which the surface on each street is determined by the grades, the traffic, the development of the abutting property, and its ability to pay. If grades permit, it is desirable to have

Mr. Lewis.

* Continued from February, 1914, *Proceedings*.

Mr. Lewis.

the same kind of pavement on a continuous thoroughfare, and it is very desirable that the driver of a team should be able to follow a pavement on which horses can travel under any weather conditions from any part of a city to any other part.

There should also be a definite plan for the location of sub-surface structures. The building of pipe subways has frequently been urged as the only means by which an orderly arrangement of underground structures can be secured and the constant opening of the street surface prevented. When a new street is being built or an old street is being widened and entirely reconstructed, the placing of pipes and conduits in subways may be practicable, but, unless all these pipes are new, and unless the greatest care is exercised to prevent leakage, especially of gas, the risk of carrying out such a plan is considerable. Paris no longer permits electric wires to be placed in the sewers, where the water and gas mains are accommodated, but separate electrical conduits are usually placed under the sidewalk. This, again, is a subject for careful engineering investigation, and is worthy of assignment to a place in an engineering organization dealing with highway work.

In the opening discussion, the city highway bureau was aptly described as the principal show-case of the city government, the pavements representing the goods in the window. It follows that there is no department of municipal work where inefficiency is so sure to result in criticism as in the highway bureau, but the organization to deal with this branch of municipal work should not be limited to the mere laying and maintenance of pavements and their cleansing; it should extend to the planning of the lines and grades of the streets, the selection of the type of pavement for each street, a rational method of financing the cost of a street improvement, and also the proper placing and care of the underground structures, which do not appear in the show-window, but which, nevertheless, are an important adjunct to every street.

Mr. Trautwine.

JOHN C. TRAUTWINE, JR., ASSOC. AM. SOC. C. E.—Mr. Durham's discussion calls attention to what may be called two systems of stratification, in the matter of highway organization: (1) The vertical system, in which each of the several geographical divisions of a large city constitutes, in effect, a little sovereign state, with its own highway organization, independent of those of the other divisions; and (2) the horizontal system, in which a single highway organization presides over the highways of the entire city, including all its sub-divisions; as Pennsylvania's blanket of "new red shale" overlies the upturned edges of many much older formations.

Now, although Mr. Durham expressed his preference for the second, or horizontal system, the vertical system appears to obtain in

Mr. Trautwine.

New York City, where the Borough of Manhattan, the Borough of Queens, the Borough of the Bronx, and the Borough of "What-not," each constitutes apparently a little highway principality, with sovereign powers over its own affairs, and permitted to act (if it chooses) in imperial disregard of the arrangements and interests of the others; so that, conceivably, two adjoining boroughs, in an outburst of local patriotism, might so revise the alignments and grades of their highways that those of one borough should have no physical connection with those of the other.

Now, to paraphrase Sterne (who has been quoted in this discussion) the speaker submits that "we do these things better in Philadelphia." Mr. Connell will correct the speaker if he is wrong; but he is under the impression that Mr. Connell reigns supreme over the highway organization of that city, and that there are no highway chiefs of Manayunk, or of Chestnut Hill, or of Frankford, or of Southwark, empowered to regulate highway matters, in their several districts, without consulting him.

Philadelphia has the horizontal system of stratification; and this is in accord with the modern tendency to centralization, with its necessarily higher efficiency.

The programme for these meetings mentions State and Municipal Highway Organizations. The speaker ventures the prediction that, within 50 years, we shall have a National highway organization, administering the streets and highways of the entire nation, and of its cities, from whatever is then the national capital; and that, within a century, we shall have a World highway organization, similarly administering, from a central office, the streets and highways of civilization —with universal benefit on a scale now undreamed of.

The same will be true, of course, of matters like water supply and sewerage, now in general conducted (on the vertical system of stratification) by each city for itself; although Massachusetts (in the advance, as usual) has set a notable example in the formation of its Metropolitan Water and Sewerage Board, which regulates these matters for Boston and its neighboring communities, each of which formerly looked out for its own interests, mostly without reference to those of all the others.

Of course, it is not to be expected that the suggestion of a national and a world highway organization will be looked on with favor by incumbents of municipal offices which, under such organization, of course, would have to accept subordination in exchange for their present sovereignty, unless indeed from those who felt pretty sure of being called to the national or world chiefship.

Mr. Goldsmith.

WILLIAM GOLDSMITH, ASSOC. M. AM. SOC. C. E.—There is one detail in Mr. Connell's discussion which has proved of vital importance to the organization of the Highway Bureau in the Borough of Man-

Mr. Goldsmith.

hattan. This relates to patrol inspection. In past years civic organizations and private citizens have complained repeatedly about the poor condition of the pavements in Manhattan Borough, and particularly with reference to the length of time elapsing between the opening of a pavement and its final restoration. These complaints culminated in an investigation by the Commissioners of Accounts during 1906 and 1907. Their report showed:

First.—That there was no system which made it possible to follow up cuts made in pavements so that they could be quickly restored.

Second.—That the responsibility was not fixed, and the shifting of blame from one to another was not only possible, but was the case whenever difficulties arose.

In 1910, under Borough President McAneny, steps were immediately taken to remedy these conditions. The Bureau of Municipal Research, working in conjunction with the Bureau of Highways, inaugurated an inspection system, which, during the last 2 years, has worked admirably. Although this is a detail in highway organization, it is vital to success, and is worth while noting.

The city is divided into districts, for each of which there is a patrol inspector. His territory covers about 10 miles of pavement, there being about 450 miles of paved streets in Manhattan. It is the inspector's duty to report at least once a week regarding the general condition of the pavements, and to report promptly on all street openings, building operations, and encroachments for which permits are issued.

In order to give a clear idea of the inspection system referred to, take, for example, a plumber's street opening. The plumber must be licensed (pass an examination as to fitness, before the Board of Plumbing Examiners), and must execute a $1 000 bond with the city for the proper performance of his work. He comes to the Permit Division, which is under the direction of the Bureau of Highways. A technical man, called the Supervisor of Permits, is in charge of this Division, and keeps in direct touch with contemplated pavement work, sewers, or other constructions; sees that sub-surface corporations do their work in streets previous to resurfacing; that all departments are notified of contemplated constructions; and approves every permit before it is issued. The plumber makes a cash deposit covering the probable cost of pavement restoration. This payment includes an inspection and overhead charge (which amounts to approximately 50 cents per sq. yd.). He must also make a cash deposit of an equal amount, which is retained for 6 months, to insure the city against any settlement of the pavement, due to improper back-fill, or other poor workmanship.

As soon as a permit is issued, triplicate inspection forms are made out on a typewriter in the Permit Division. These forms provide space to give the inspector in the field all the information necessary for a

proper inspection of the work. These forms are called: (1) a "Preliminary"; (2) a "Tickler"; and (3) a "Final". Mr. Goldsmith.

The "preliminary" report is given to the inspector, whose duty it is to see that the conditions of the permit are complied with, to report thereon what work was done, and, by actual dimensions and sketch, show just how much pavement was disturbed, and the quantity to be restored and its exact location.

The "tickler" and "final" forms are filed in a cabinet provided for that purpose in the main office. When the "preliminary" report is received from the inspector, the "final" is sent to the pavement contractor, who honors it as an order from the city to restore the pavement. The contractor hands this "final" to his foreman, who takes it out on the work and makes the necessary repairs in amount and location as shown.

With these repair gangs there is another set of inspectors, called maintenance men, who are entirely independent of the patrol inspectors. The foreman of the repair gang hands this "final" report form to the maintenance inspector assigned to his work, and it is this inspector's duty to see that the pavement, of the exact amount and location shown on the inspection form, is repaired according to specifications. He then records the actual amount repaired and signs this "final" form in a space provided for that purpose. These maintenance inspectors are notified by a chief inspector, about 18 hours in advance, just where their work will be on the following day.

After the day's work is completed, the inspector fills out and signs the "final" reports, and mails them to the main office. In this way, an independent check is obtained, and collusion is almost impossible between the inspectors and contractors as to the area of pavement to be restored. The paving company then presents its bill for this restoration work to the city, and a clerk in the office, uses the "final" reports as a basis on which to approve the bills.

During 1913 an experiment on patrol inspection was tried, the object being to reduce the number of patrol inspectors and put them where the work was urgent. The territory north of 59th Street and west of Fifth Avenue, in Manhattan Borough, was patroled by one inspector in an automobile. Previously, this section had been patroled by twelve men. The difference in cost was:

12 Inspectors at. .	$1 200 = $14 400
as compared with:	
1 Inspector at. . .	1 800
1 Chauffeur at. .	1 200
Maintenance and depreciation of automobile	1 000
Total	4 000
Yearly difference, or saving.	$10 400

Mr. Goldsmith.

This covered a mileage of approximately 132 miles and gave a patrol inspection cost per annum of $30 + per mile, as compared with $109 + per mile by the old system, showing a saving of $79 per mile per annum, or a total of $10 400 per annum.

Although this scheme worked out well in the district where it was used, it probably would not be as effective in heavy traffic sections, where the territory covered would be limited on account of congestion and where the speed of the automobile would not be an advantage.

The inspection and follow-up system referred to, which includes a number of patrol inspectors and an independent number of maintenance inspectors, has the following advantages:

(1) It places individual responsibility on the inspector on the work, as the reports must be signed by him;
(2) It makes the shifting of responsibility impossible;
(3) Possible collusion between inspector and contractor is out of the question;
(4) Experience shows that better results are obtained.

The fourth statement is easily verified by a comparison between the present state of the streets in the Borough of Manhattan and their condition under the old system.

In a highway organization, the details of the inspection system are vital to a successful administration, because the success or failure of highway work depends on the field inspection. The speaker believes that the system outlined is feasible, covers all necessary requirements, and, if properly supervised, will get rid of many of the difficulties with which a city highway engineer must contend.

Mr. Ricker.

GEORGE A. RICKER, M. AM. SOC. C. E.—The speaker is glad of the opportunity to say a few words about the reorganization of the New York State Highway Department, and the methods that were followed in selecting the Division Engineers. When Commissioner Carlisle and the speaker undertook this reorganization there were only three regular Division Engineers on the work, and, as the new Highway Law commanded the redivision of the State into nine divisions, there were six vacancies to fill. It was somewhat troublesome to think of how to secure competent engineers for these divisions who, at the same time, would be good executives, for it must be borne in mind that these divisions are very large, each containing a mileage of roads as great as the whole of Massachusetts.

It was thought to be practicable to secure by civil service examination and appointment men who had the necessary personality, men with executive force and the ability to command. By permission of the State Civil Service Commission a plan of co-operation with the Highway Department was established in the preparation and conduct of an examination, which consisted of three parts. First, an oral

Mr. Ricker.

test (and there is point in its being placed first); second, two theses, one on the subject, "Essentials of Highway Construction," and the other on any large work in which the applicants had had experience; the third section consisted of a sworn statement of each man's experience.

There were about 160 applicants. One by one the men were called into the office of the Civil Service Commission on a given day, and two members of the Board of Consulting Engineers and the speaker sat with the Commission. An attempt was made to put the candidates at ease by the attitude of the examiners, and by asking them unimportant questions until any nervousness which they felt was overcome. After a few minutes' conversation the leading questions, calculated to bring out the characteristics of the candidate, were put. It was assumed that the candidates were ready with technical matter, but technical questions were avoided, as the examiners were quite sure that the men were well equipped along that line, and if not, that would be found out in the later tests.

The object of placing the oral test first, it will readily be seen, was because men of executive force and strong personality were needed. They might have all the technical knowledge in the world, all the education and experience necessary, but, if they could not control men, they would not make good Division Engineers. By this test it was possible to exclude from the examinations that would follow men who were lacking in necessary personality to make good executives. Some fifty or sixty men were selected, whom it was thought were very desirable, and they were instructed to enter the other tests. It should have been stated, by the way, that as each man left the room the examining board agreed on his mark before the next man was called.

The entire board read and criticized the theses, and, as a result of the complete examination, the Civil Service Commission presented a list from which six men were selected from among the first seven men: one from the Board of Water Supply, three from the Barge Canal organization, one from the Highway Department, and one who had had six years' experience in the Panama Canal work. These six men, with the three from the old organization, constitute the Division Engineers of the reorganized Department. All these men are members of this Society, and all are also graduates of recognized engineering schools, the several institutions represented being the Massachusetts Institute of Technology, Cornell, Rensselaer, Polytechnic, and Marietta College (Ohio).

Following this start in the reorganization, each of the Division Engineers has been supplied with an Assistant Division Engineer, the latter from promotions in the Department. An examination was held, following a plan similar to that outlined for the Division Engineers, and twelve men from the first thirteen were appointed, some divisions getting two men on account of their size and the magnitude

Mr. Ricker. of the work. These Assistant Division Engineers, technically known as Resident Engineers, were taken, with one exception, from the top of the list down. The one man passed, as in the case of the Division Engineers' list, was not passed on account of any political reasons. Politics had absolutely nothing to do with the selection of any of these men. They were chosen and appointed before any one of them knew that they were being considered, so that nobody had any chance to "see the Commissioner" or the speaker regarding them, and they had no opportunity to send anybody on their behalf if they had wanted to, and, as far as the speaker knows, none of them ever thought of doing such a thing. Thus it is seen that the reorganization is purely on a merit basis.

Arrangements are now being made for the selection of Assistant Engineers to place in charge of counties. The Division Engineers have recently met and discussed the list of available men presented by the recent promotion examination, and have agreed among themselves as to the disposition of these assistants throughout the State by counties.

An attempt is being made to establish something approaching a military organization, and the Division Engineer in complete charge of his division will be supported and sustained. Politics will have absolutely nothing to do with the work if it can be prevented, and it is thought that it can. Commissioner Carlisle, although not an engineer, has engineering sense, and in a marked degree possesses executive and administrative ability.

In January, 1914, there was established a system of weekly reports, which are really time sheets, so that every man in the Department reports to the Bureau head and Bureau heads send to the Department head a record of what every man is doing every day. Thus it is known where the men are, and what they are doing, and in that way one can readily keep track of the force.

FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

Mr. Tillson. GEORGE W. TILLSON, M. AM. SOC. C. E.—This subject is of great importance; it has been discussed for a great many years, and will be discussed for many years to come. The speaker has made the statement several times that in the preparation of plans for road and street work engineers were more behind than almost any class of constructive workers. That statement will have to be modified from time to time, as the subject is receiving so much discussion from year to year and from day to day.

It will be remembered that Mr. Green started out with a proposition that what was said in the discussion must be new. That is a

very difficult requirement, and, under it, the speaker feels that he cannot qualify, especially after listening to Mr. Green's discussion; he thinks that a man who could talk for ten minutes and say nothing but what was new to every member would be a wonder. If the speaker can suggest one idea that is new, he will be satisfied. The point he wants to make is that the ideas which govern roadway and street construction and the design of both must not be said once and then laid aside. In order to make inspectors and road supervisors entirely familiar with them, they must be reiterated, told over and over again, said in a new way, or said in the same way, until they know them as thoroughly as they know the alphabet. Mr. Tillson.

In the design of work of this kind, it seems that the first thing an engineer must do—and the whole situation will be discussed in as general a way as possible—is to make himself thoroughly familiar with the work that is before him. If an engineer is to design a bridge, if an architect is to design a building, the first thing to do is to find out what will be required of this bridge or this building. The first thing the engineer must do is to study the requirements of the road he is to build or the pavement he is to construct.

In doing that, he has a great many things to consider. That is brought out to a great extent by Mr. Green. The character of the traffic: Until the last few years no one has considered it necessary, or, if necessary, that it was worth while, to have a traffic census, either in city streets or country roads. No longer ago than December, 1912, at the Road Convention in Cincinnati, after an elaborate paper on "Traffic Census," and what was to be gained by it, a State engineer said, "Well, after all this traffic census, you have got to use your common sense." That is true. That is the very object of having the traffic census, so that common sense may be used intelligently.

Now, the traffic is not the only thing that must be studied. One must study where the road is to be laid, what its objects are, the character of the different traffics, and, another thing, not only the entire traffic, but how that traffic is to be applied.

The engineer of the Borough of Fulham, London, has calculated that, under a certain traffic which has been observed, a wood pavement will last for a certain term of years; and that, when a wood pavement is laid on any street, he can calculate its life in accordance with the traffic which is to be applied. That must be considered to a great extent in a way which the speaker will mention later.

Now, after one understands fully the requirements of the road, the next thing is to find out the character of the materials available for use. This part of the question was not nearly as important 30 or 40 years ago as it is to-day. At that time, stone in its various forms was practically the only material used in road or street construction. Now, there is not only stone, but brick, wood, and the bituminous

Mr. Tillson.

materials, the latter laid in all their various forms and composed of the different kinds of bitumens.

The engineer, in studying this phase, must consider first the character of the stone. He must know what the different kinds of stone will stand under the different kinds of traffic. The speaker was severely criticized once in college when he said—in considering the abstract question of foundations—that stone made the best foundations. His professor contended that the statement was absolutely wrong, because, under certain conditions, sand is just as good a foundation as stone. Now, under some conditions of traffic, a cheaper stone is just as good as a dearer, so that, though in a certain way one stone would be better abstractly, the other stone might be just as good concretely.

It is the same way with other materials. The engineer must understand brick and the different grades of brick; he must understand wood, what the different varieties of wood will do, what kind of treatment the wood requires; and then, when it comes to the bituminous pavements, he is uncertain at the present time. It does not make any difference where he goes, he will get different advice, different information. The information may be true, for the reason that different people are studying different kinds of bitumens, and they know more about the bitumens that they are studying than about any others, and while they are giving perfectly good and true information about their own, it seems to be contradictory to what was received before, because they did not understand the values of these other bitumens.

Now, let it be assumed that the engineer understands thoroughly the condition of his road, and all the different qualities of the materials, both in their crude state, and also how they can be mixed in order to produce satisfactory results.

The next thing is to make the application, and here there should be elaborated what was said about the traffic and the wearing out of the pavement in a certain length of time. Take, for instance, the assumption that a traffic of 60 000 000 tons will wear out a pavement, and the road will receive 2 000 000 tons of traffic per year. The logical deduction would be, then, that the pavement would last 30 years. Not necessarily; because there are certain materials in use which are affected chemically by climate and by the natural action of the atmosphere.

Now, as was stated by Mr. Green, wood and the bituminous pavements do require a certain amount of traffic in order to give the best results. So that, even if there exists a traffic which, theoretically and mathematically, would wear out the pavement in 30 years, this traffic may be so scattered that it would not be such as would be most economical and best for the pavement; and, on account of climatic conditions, it might be destroyed by disintegration and decay before it was worn out. Brick and stone are probably the only paving materials

Mr. Tillson.

now in use the life of which can be determined according to the foregoing principles, unless the traffic is so heavy that it will wear out the pavement before the materials decay or become disintegrated. The best illustration of this is in the use of wood. If these 60 000 000 tons of traffic will be applied to a pavement before the wood would naturally decay, there is no necessity for treating the wood with any material that will prevent it from decaying. That principle is recognized in Europe, where, in the great majority of cases, the wood will wear out before it will rot out, and it is only superficially treated. In Paris, where they use soft wood, and where the wear of the traffic in certain streets is such that the pavement will last only 7 or 8 years, they use what might be called no treatment; that is, the blocks are immersed for perhaps ½ hour in creosote oil and then laid in the streets.

Paris, last summer, was erecting a plant in which the blocks will receive a treatment of 10 or 12 lb. per cu. ft. That is not necessarily to keep the blocks from decaying, but to prevent them from swelling and causing the pavement to bulge. That is the main thing that they hope to gain by increasing the quantity of preservative to 10 or 12 lb. It can readily be seen that it would be wrong to use an elaborate form of treatment to prevent decay when the block itself will wear out before it would rot out.

In America, where there is not this intensive traffic, and where a pavement is expected to last longer, it is an entirely different proposition. Take a certain street, for instance, in Brooklyn, where wood block has been laid about 10 years. Practically no repairs have been made, and if the traffic continues as it has in the past 10 years—and being a residential street, it probably will—that pavement will not be worn out in 50 years from the time it was laid. So it will be seen how necessary it is to lay treated blocks under such traffic, so that they will last as long as they can, as far as decay is concerned; and for that reason it makes a difference whether the amount of traffic that will wear out a pavement is scattered over 4 or 5 years or 25 or 30 years.

There is just one little point more, which was referred to by Mr. Green and also to a certain extent, by Mr. Whinery. All these different principles that have been spoken of can be settled in the laboratory; but, if an engineer determines on granite, for instance, for a certain street, and writes to his client, who may be 400 or 500 miles away, and tells him of his decision, the client might write back and say there is not a granite quarry within 1 000 miles. The point to be recognized is the question of availability. That is of the utmost importance, and the speaker has often thought that, in the possibility, or rather the power, of Nature to furnish everything necessary for the development of the world, the development of civilization has been shown in the pavement line as much as in any other way.

Mr. Tillson.

In the Central West, where there is no rock suitable for pavement, where all outside materials can only be brought in at great expense, Nature has furnished a material—clay—from which can be, and is, manufactured a brick that gives satisfaction and is almost as durable as the best of stone itself.

In summation, let it be said that if an engineer, who has roads or street pavements to plan, has first a knowledge of the conditions which his pavements must meet, if he understands thoroughly the qualities of the available materials, and if with that knowledge he makes his application in accordance with sound common sense, taking into consideration the atmospheric conditions and the availability of materials, he will undoubtedly reach a conclusion that will give satisfactory results.

The following question has been asked: "What conditions would be considered, and would govern, in making a selection of asphalt blocks, brick, or bituminous concrete for a street, say, in Brooklyn?" This is easily answered, because, as a matter of fact, Brooklyn uses only one of the materials mentioned, *viz.*, asphalt blocks. Brick has not been used because it is expensive and makes a more noisy pavement than asphalt, so that the latter is more desired by the property owners. Asphalt blocks are generally selected in preference to asphalt where the grades are comparatively steep. The blocks are less slippery than sheet-asphalt, as they are made of coarser materials, and on account of the joints between the blocks. Brick would be selected for a street on which the traffic would be too heavy for asphalt block. The speaker has never used asphaltic concrete, and cannot give an opinion as to whether this material would give a better wearing surface than asphalt blocks.

Mr. Meeker.

ROBERT A. MEEKER, ESQ.*—The speaker cannot accept the theory of harmonic waves, said to be due to traffic, nor the beautiful system of harmony of the spheres, for, from practical experience, it has been learned in his Department that it is very necessary indeed to watch the output of the mixer carefully. The proportions of stone, of sand, and of asphalt may be all right, but the first run of the mixer will be too rich in asphalt, the middle of the run will be just right, and the latter part of the run will be too lean; consequently, one would have a pavement composed of, for example, 12%, 8%, and 4% of asphalt, all from the same mixer, notwithstanding that the stone was weighed and measured carefully, as was the asphalt, before either was put in the mixer.

This mechanical failure is often overlooked. It caused so much trouble that the speaker was led to make a careful investigation of the run of the mixer, and it was found that, in many cases, the asphalt concrete which had been condemned as not being properly mixed,

* State Highway Engineer of New Jersey.

owed its lack of uniformity, not to the workmen who were in charge (they had done their duty faithfully and carefully), but to the machine. Mr. Meeker.

In connection with this theory of harmonic waves, there was one very important point made by Mr. Washington which is worthy of more than passing notice, namely, the irregularities caused by improper rolling. The speaker has had considerable trouble in the past, even with water-bound macadam roads. When the causes of the so-called waves are carefully analyzed, it will be found that the trouble lies, not in the traffic, but in faulty construction. The first macadam roads built in the United States were rolled with heavy three-wheeled rollers. There never were any waves in those roads. Later, an endeavor was made to reduce the cost of construction, and, to that end, horse rollers were substituted for the heavy English steam rollers; still later, different types of steam rollers were used, and, last of all, the tandem roller was brought forward as the best. The speaker tried them, and had macadam roads with harmonic waves long before automobiles were used. The real reason for these waves, or hills and hollows, in the road, may be found in the method of construction. If the base is not thoroughly compacted, not thoroughly rolled, and not brought to an even and uniform surface, then, if those waves do not appear at once, they will develop eventually. When to this wavy base is applied bituminous concrete which is not uniform in composition, two sets of waves—two sets of hills and hollows—will appear, not due to traffic, but to careless construction.

Highway engineers should not try to dodge their responsibility for poor workmanship by charging it to motor vehicles or to the rhythm with which they travel over the highways, when the fault lies in the manner of construction. This must be carefully watched. Careless construction was one of the reasons for the failure of so many penetration roads. The speaker has had experience with penetration roads in which he was told that the distributing wagon had stood for a time at a certain point and that the hump of bitumen, seen on top of the road, was due to the fact that the controlling valves were leaking.

In one case, coming under the speaker's immediate observation, he knew that the wagon had not stopped within 100 ft. of that particular point; therefore, this explanation was evidently wrong; hence, a series of investigations was made, whereupon it was found that the percentage of voids in the road varied greatly. Where the road was comparatively dense, the bitumen appeared in humps on top of it and disappeared entirely at a depth of 2 in., and where hollows appeared it was found that the bitumen had penetrated in many cases to the depth of 7 in. The same quantity of bitumen per square yard of surface having been applied, there was consequently a recurrence of the fat and lean

Mr. Meeker. bituminous mixture encountered in the output of the mechanical mixer. A careful analysis of numerous faulty spots, in both broken-stone and bituminous pavements, has convinced the speaker that the varying percentage of voids is a fruitful cause of many so-called failures; in other words, a lack of homogeneity in the mass is the real cause of the wavy surfaces so often seen.

Mr. Beaty. R. E. BEATY, ASSOC. M. AM. SOC. C. E.—These remarks may not have a literal application to the discussion of "Factors limiting the selection of materials and of methods in highway construction", but, with regard to the use of wood paving blocks, the speaker is of the opinion that some mistaken ideas are abroad which should be corrected.

In European practice it has been customary to use a very light treatment in preparing blocks, that is, to treat them with 10 lb. of oil per cu. ft., or in some instances merely to dip them into the preservative.

The wood principally used in France is known as "Landes" pine; in England "Baltic deal" is used. An examination of some of these blocks shows very uniform wear, and that the wood very much resembles southern short-leaf, yellow pine, with the difference that the wood is much more nearly uniform in density and strength of fiber.

Tests made by the Borough of Manhattan, and by the Department of Forestry, of the United States Department of Agriculture, show very clearly that although some American pines very much resemble the timber used in France and England, they are of much less uniform quality. The characteristics of southern pines will be considered in more detail later.

In the speaker's opinion, it will be taking a grave risk to try to follow the European practice of using a light treatment of oil. This method will have the effect of leaving some of the blocks in the pavement untreated. This condition is indicated to some extent in foreign countries, where numbers of blocks have to be removed on account of rotting.

All the speaker's observations of experiments and—what is of more value—of the actual condition of 250 000 sq. yd. of blocks after treatment, in charges of from 400 to 500 sq. yd., has convinced him that, as the quantity of oil is decreased below 20 lb. per cu. ft., it becomes increasingly difficult to get either a uniform distribution of oil between the individual blocks making up the charge, or a satisfactory penetration into all the blocks. This difficulty can be overcome to some slight extent by the slow application of the oil, but this is not commercially desirable, for the reason that lengthening the time of treatment would often curtail the output of plants fully 33 per cent. Attempts at light treatment have been observed many times. In

Mr. Beaty.

one case, 16 000 sq. yd. were being treated with 16 lb. per cu. ft. The specific gravity of the oil used was 1.06 at 38° cent. At first a 16-lb. treatment was tried, but it was found necessary to increase this to 18 lb., because many of the blocks, when the lighter treatment was applied, showed a penetration of only ¼ in. from the surface. The inference is, that in injecting 16 lb. or less into the blocks, the duration of time the pressure is applied to the treatment cylinder is too short to allow the pressure to be raised to a height sufficient to cause oil to penetrate the denser and harder blocks before all the oil is used up. In experiments reported in December, 1912, on various samples tested for the Department of Public Works, Manhattan, it was shown that the quantity of oil taken up per cubic foot by the blocks, under a pressure of 150 lb. per sq. in., which pressure was necessary to inject an average of 20 lb., varied widely among the different species of southern yellow pine blocks making up the samples. Some blocks received the equivalent of 28 lb. per cu. ft., and others received only 12 lb. At less than 50 lb. cylinder pressure, 10 lb. can be injected. If thus treated, many blocks in a charge would receive practically no oil. This lack of uniformity in material, though not so marked in the "Landes" pine and "Baltic fir", indicates a cause for decay in individual blocks.

There is in very general use, by some wood block manufacturers, the term "Southern Yellow Pine" to describe the timber used in manufacturing their product. This nomenclature, as used in the 1913 *Proceedings* of the Association for Standardizing Paving Specifications, is intended to cover all the five or more varieties of pine found in the South, and it was so stated in the "discussion" at the time of their adoption. That the term is misleading, indefinite, and inaccurate is readily seen by a reference to Bulletin No. 13 of the Division of Forestry, of the United States Department of Agriculture, in which five species of pine, several of them occupying large and definite areas of the Southern States, are given, as follows:

"Long-leaf" pine (*"Pinus palustris"*, Miller);
"Cuban" pine (*"Pinus heterophylla"*);
"Short-leaf" pine (*"Pinus Echinata"*, Miller);
"Loblolly" pine (*"Pinus taeda"*, Linn); and
"Spruce" pine (*"Pinus glabia"*, Walt).

These species vary widely in physical characteristics, such as average percentage of resin, number of annual rings, weight per cubic foot, average percentage of heart wood, strength in compression of fiber parallel to the grain, etc. Two of these definite groups will be compared: the "Long-leaf" and the "Short-leaf" yellow pine. Tests

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by the U. S. Department of Agriculture showed the following relative values:

	Long-leaf.	Short-leaf.
Strength in cross-breaking.....	10 900 lb.	9 230 lb.
Strength in compression (per square inch):		
Average lowest.....	5 650 "	4 800 "
Average highest.....	6 850 "	5 900 "

In summarizing the results of tests on various pines the Department expert states: "From these results, though slightly at variance, we are justified in concluding that Cuban and Long-leaf are nearly alike in strength and weight, and excel Loblolly and Short-leaf by about 20 per cent."

These values were checked by experimental tests, at Columbia University, on samples secured by the speaker for the Borough of Manhattan, to which reference has previously been made. The results of these tests are not given here as the tests were made to demonstrate the value of a requirement for a minimum number of annual rings in timber used in the manufacture of blocks. It is interesting to note, however, that the average strength of three samples of long-leaf was more than 10 000 lb. per sq. in. in compression, parallel to the fiber, and some samples of short-leaf were as low as 5 600 lb. per sq. in.

Other tests, made at this time by the Highway Department, indicated clearly that "short-leaf" pine, as compared with "long-leaf", has a greater range of expansion and contraction, and is more difficult to impregnate uniformly with preservative. Each of the species considered has its uses, but, as they often vary widely in price, as well as in physical characteristics (as has been shown), then, for the mutual protection of the bidder and the ultimate user, engineers should specify clearly the kind of timber to be supplied.

In the matter of the limitation mentioned by Mr. Tillson, the speaker heartily concurs. It would not be reasonable to conclude that, simply because "a block might possibly wear out before it would rot out", it would be wise to use a very light treatment.

It would be necessary to use a certain quantity of oil in order to protect the blocks against bulging or "buckling". The proposition to use untreated timber is also unwise, because it has been observed that, if certain varieties of pine, if untreated, are exposed to the weather, they will be destroyed by fungi and other destructive agents in less than 4 years.

Mr. Meeker's remarks with regard to proper proportioning and mixing are extremely important. Several engineers of the Bureau of Highways, Manhattan, have endeavored for the past 2 years to restrict the use of machine mixers to those of the "batch" type. It

has been noted that many "continuous" mixers furnish a mix which is often too "lean", and at other times practically devoid of stone. Mr. Beaty.

If continuous mixers are not considered safe for use on such structures as the Panama Canal, the Keokuk Dam, the New York Subway and the Aqueduct work, their use should not be permitted for mixing concrete for use in foundation for pavements designed for heavy traffic.

WILLIAM M. KINNEY, JUN. AM. SOC. C. E.—Referring to concrete pavements in which there appears to be some pulverizing action under the bituminous carpet, the speaker has made quite a number of investigations of pavements of this type and, in the few cases where such condition exists, has found that the surface of the pavement, prior to the application of the bituminous carpet, was of poor quality, either because of the use of poor aggregate, or because the surface of the concrete had been frosted. Mr. Kinney.

In floating concrete made from an aggregate in which there is an objectionable quantity of dirt or loam, this unsatisfactory fine material is brought to the top of the concrete and makes a surface which is of poor strength and to which another material does not readily bond. A skin on the concrete thus formed might show the pulverizing action to which Mr. Crosby has referred.

When the concrete has been frosted prior to the application of the bituminous top, practically the same result would be expected, as the thin layer loosened from the concrete by the freezing action would pulverize rapidly under travel.

When properly made concrete is covered with a bituminous carpet, and this carpet later peels off, there is usually very little material such as sand, gravel, or mortar clinging to the bituminous material.

It would be interesting to know the location of the roads to which Mr. Crosby refers, and to ascertain whether a report on the character of the aggregate and time of doing the work is available.

WILLIAM GOLDSMITH, ASSOC. M. AM. SOC. C. E.—Speaking about harmonics and what Mr. Washington has said, the speaker would like to relate one little incident which happened on Second Avenue, in New York City, which brings out that point rather clearly. Mr. Goldsmith.

From 1st to 23d Streets, on Second Avenue, experimental pavements have been laid and on one stretch, between 15th and 17th Streets, the pavement is of asphalt block. This section is in itself an experiment, different qualities of blocks being used within the area. One part of it is laid with soft blocks having a large percentage of bitumen. Other sections are laid with harder blocks.

About a year after the pavement was opened to traffic, a number of depressions and holes appeared, and the wave effect which has been mentioned was also evident. It became necessary to replace these

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defective parts, and in ripping up the old blocks, a curious condition was found. The asphalt blocks which were lined up perpendicular to the curb had taken a semicircular form; they had moved up on one another under the heavy traffic, so that waves were formed. Originally, the blocks were 5 in. wide, 12 in. long, and 3 in. deep. When removed their depth was found to vary from a minimum of 1 in. to a maximum of about 6½ in. The width and length had correspondingly increased or diminished, showing that there was little if any wear of the material, but that a distortion of the blocks had taken place. This showed conclusively that the asphalt moved or crept up and worked into semicircular shape, because the blocks were soft, and not on account of rolling. Adjoining these particular blocks were some of a harder nature which retained their shape, thickness, and alignment.

Approximately every 6 ft., and perpendicular to the curb, steel plates were embedded in the concrete foundation, extending up into the asphalt block joints. The object of these plates was to prevent the creeping and distortion of the blocks, which was anticipated. The plates, however, took the same semicircular shape with the blocks, the pressure evidently being great enough to detach them from the concrete in which they were embedded.

There was no grade on that part of the street where these blocks were laid. The traffic, both light and heavy, moved in the direction of the curves, and the latter had a middle ordinate of about 7 ft., the chord being about 20 ft.

This incident seems to show that in this case the wavy effect was due to the softness of the material under heavy traffic, and not to any other cause.

There is another point, which has not been spoken of, but is of great importance. It is the factors which limit the selection of pavement to be used in the railroad area in city streets—asphalt, wood block, granite, or other material used in city streets. Many streets have car tracks in the center. The question always arises, shall the same pavement that is used in the roadway be placed within the railroad area?

According to the usual railroad laws and charter requirements, the pavement between the railroad tracks and for 2 ft. on each side of them, is maintained by the railroad company, but must be constructed in accordance with specifications prepared by the city engineer.

Investigations in New York City have shown more depressions and other defects on pavements between the tracks and near them, than at other places. The question, therefore, naturally arises, which is the best pavement to place in the railroad area? Asphalt, wood block, or granite?

The consensus of opinion in Manhattan Borough seems to be that the improved granite block is the most satisfactory. It has some dis-

advantages, however, on account of the ties which come in between the slot rail and track, which make it necessary to chip the granite blocks to make them fit well between these cross-ties. Mr. Goldsmith.

The joints, in Manhattan Borough, are made with paving pitch and sand. The speaker is informed that the street railroad companies in Brooklyn have used cement grout almost entirely during the past 8 or 10 years, and very successfully.

This subject is brought up with the hope that some one will discuss it and give his experience with relation to pavements in the railroad area.

GEORGE A. RICKER, M. AM. SOC. C. E.—In reference to this subject of materials some curious things have happened in the New York State Highway Department, as disclosed by recent investigations, and steps are being taken to try and correct mistakes that have been made. Mr. Ricker.

Roads were designed by some of the engineers who had never seen the territory in which the road was located. Specifications were applied to sections of which the engineers were totally ignorant of conditions. The speaker found places where solid rock was being excavated, and the road replacing it made of concrete, and where bituminous roads were built on blow sand foundations. At present the Division Engineers are required to send in, with their plans of roads to be improved, a report of their personal inspection after walking over them.

The Department has also instituted a survey of the State to locate available materials. One of the Assistant Engineers, who is somewhat of a geologist, and has been engaged in the Testing Bureau for some years, has been assigned to the work of examining materials in all parts of the State, and a chart is being compiled as a record of his examination.

D. B. GOODSELL, ASSOC. M. AM. SOC. C. E.—The speaker cannot say much about "harmonic waves" in asphalt or bituminous pavements, but readily agrees with what Mr. Washington has said regarding Mr. Crompton's ideas as to rolling. He believes that waves such as mentioned by Mr. Crompton might occur in some hard homogeneous material, but, with one as plastic as a bituminous pavement, such waves are likely to be due to two causes: lack of cross-rolling, and lack of homogeneity of the material. Most highway engineers are familiar with the effect of cross-rolling, and the speaker believes that, if roads generally were rolled crossways, as well as longitudinally, the regular waves would not occur as frequently. Mr. Goodsell.

No doubt the uniformity of the material, or in other words, the number of voids in it, has a very great influence on the evenness of wear; and the secret of obtaining a good road, other things being equal, seems to be the even distribution of voids throughout the material.

Mr. Goodsell. Attention is called to the measurements of wear of pitch-macadam pavement made by Mr. John Brodie, City Engineer of Liverpool, which seems to be the only instance of systematic measurement of wear under known tonnage of traffic. Some method such as he uses seems to be desirable for the determination of the relative life of the various kinds of hard pavement, including asphalt block. The speaker is not informed of any measurements which show the relative wear of asphalt-block pavements, and believes that this subject is an interesting field for investigation.

Mr. Durham. H. W. DURHAM, M. AM. SOC. C. E.—Reference has been made to stone block pavements in the car tracks, and to the different methods of filling joints. The speaker thinks that one important point in New York City practice was not brought out as clearly as possible. It explains very largely the difference between the practice in the Boroughs of Brooklyn and The Bronx and that adopted for Manhattan, and is mostly due to the radical difference in the types of track construction. In Manhattan an underground current-conductor with central slot between the rails is used almost entirely. In all the other Boroughs current is taken from an overhead wire, necessitating no special work in the track. The Manhattan construction requires the possibility of very frequent access to the substructure, for repairs either to the current-conductor system or the cast-iron yokes carrying the track. Therefore, it has been found more desirable to use a type of pavement which can be easily removed and promptly restored after repairs are made.

The use of a cement grout as a filler gives an excellent surface between the rails, as may be seen on some of the Brooklyn and Bronx streets; but if this were used in the Manhattan track construction it would render very difficult the making of any openings for repairs to the numerous hand and manhole boxes and other sub-surface structures, and it would also be difficult to obtain proper setting of the grout filler after repairs were made. This largely explains the difference in practice.

EQUIPMENT AND METHODS OF MAINTAINING BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

Mr. Spencer. HERBERT SPENCER, ASSOC. M. AM. SOC. C. E.—Any discussion of a subject such as the equipment and methods for maintaining bituminous surfaces and bituminous pavements must, of necessity, be limited by the length of time such surfaces have been in existence, and should properly include: (1) classification of the surfaces and pavements required to be maintained; (2) methods and material used in their construction, together with the causes responsible for their failure; and (3) equipment and methods required for their maintenance. The subject being of such importance, and having such possibilities of detail,

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the writer is of the opinion that a summary of these headings is all that should properly be attempted in a limited discussion of this nature. It must also be remembered that bituminous surfaces and pavements of various kinds having a large mileage, have ended their period of usefulness, and their maintenance now becomes a matter of first importance and requires the development of machinery and methods for their economical upkeep. These can come only from actual experience, and the lines along which such development should follow is all that can be attempted at present.

During the past 5 years many types of bituminous surfaces and pavements have been placed on the public streets and highways. Some of these have attained sufficient merit in the public estimation to warrant their continuation, and others have shown such faults in their original design and construction that their further development was not considered economical or wise. Without doubt, new types of bituminous surfaces and pavements will be exploited with the growth of highway work, but it is felt that these cannot deviate very much from the existing types, which are defined by the Special Committee on Bituminous Materials for Road Construction as consisting of superficial coats of bituminous material with or without the addition of stone or slag chips, gravel, sand, or material of similar character. This definition may include: (*a*) treatment of gravel or stone roads with cold bituminous material; (*b*) treatment with hot bituminous material; or (*c*) treatment of a Portland cement concrete base with either hot or cold bituminous material.

The Special Committee on Bituminous Materials for Road Construction defines a bituminous pavement as "One composed of stone, gravel, sand, shell, or slag, or combinations thereof, and bituminous materials incorporated together by mixing methods." This definition may include: bituminous concrete, laid either hot or cold; bituminous macadam (penetration method); bituminous mortars, consisting of bituminous cement and fine material aggregate, such as rock asphalt, and laid either hot or cold; sheet-asphalt, asphalt block, and similar pavements

The maintenance of bituminous surfaces and bituminous pavements is measured largely by the methods and materials used in their original construction. Although it is not considered necessary to enter into a discussion here of the specifications for any of the types just mentioned, the generally accepted limitations of methods of construction may be summarized as an index to the extent of maintenance.

For bituminous surfaces, the thorough sweeping of the base is of great importance for applications of hot bituminous materials, and should be resorted to for cold applications wherever an extremely dusty condition of the surface is found. The bituminous materials used range from asphalt oils and tars to the heavier products, of like char-

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acteristics, requiring heat for their application. Of importance is the selection of the cover. For the hot applications, coarse sand or grits, preferably washed, are commonly used for automobile traffic; but, for steep grades and horse-drawn traffic, stone screenings are better. The items, therefore, influencing the success or failure of bituminous surfaces include the preparation of the base, the selection of the bituminous material, the selection of the cover, and, of equal importance, the machinery to be used in the application of the bituminous material. The treatment of cement concrete surfaces with bituminous material is of too recent origin to warrant an opinion of probable causes of failure, but may be due to any of the following: (1) base floated to a smooth surface, precluding possibility of any bond with bituminous material; (2) bituminous material lacks necessary characteristics to enable it to adhere to concrete; (3) base too hard and surface too thin to resist impact of horse-drawn traffic and raveling of surface results.

The danger of failure of bituminous pavements is lessened when care is used in the preparation of specifications and intelligence is shown in the inspection of the work. When failures do occur, they can generally be traced to unsuitable materials, poor foundations, improper machinery, or inferior workmanship. Bituminous concrete mixtures depend to a large extent on their structural stability, which is determined by their density, and by the continuous adhesion of the bitumen-covered particles under traffic, and violation of such well-known principles as grading of the mineral aggregate, consistency of bitumen used, or temperature of mixture and manner of laying, will lead to failure. For bituminous macadam pavements constructed under the penetration method, the quality of the stone, the proper sizing of the stone for each successive course, and the consistency of the bitumen used often determine the success of the pavement. These features were discussed by the writer in the 1912 Road Meeting of the Society.* It is not considered advisable to enter into a description of the causes of failure of sheet-asphalt, asphalt block, and similar pavements at this time, as this subject is fully covered in the discussions of Societies giving particular attention to these classes of pavements.

Bituminous surfaces originally treated with cold bituminous material may be renewed at slight cost. The extent of sweeping such surfaces is governed by the area and character of the existing cover on the road. If all loose dirt is swept from the original surface, and the bituminous material is covered with clean sand or screenings of a character not readily ground up by the traffic, then a re-treatment without the necessity of sweeping may safely be attempted. Roads properly cleaned, oiled, and sanded should leave a coating of bituminous material

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 615.

to which the fresh material can readily attach itself. The quantity need not exceed ⅜ gal. per sq. yd., and if sand is required, not more than 10 lb. per sq. yd. should be used, or a mushy, soft condition will follow. The bituminous material should be of such consistency that it can be readily applied by a pressure distributor, and should contain a minimum of material likely to lubricate the stone base. Where more than one application per season is required, the road should be treated in the fall, in order to assist as much as possible in maintaining the surface through the winter. Mr. Spencer.

The most successful hot oil or blanket treatment, in the writer's opinion, is that which has been done in Massachusetts; and, on many miles of roads which he has been over in that State the problem of maintenance has been solved successfully. Mr. Farrington has carefully gone into the methods of maintaining this type of surfacing, and detailed descriptions, extending over a period of 4 years and given in previous publications of this Society by Messrs. A. W. Dean, F. C. Pillsbury, and J. A. Johnston, should serve as a model in those communities which really desire economical maintenance on roads subjected to heavy automobile traffic. For roads treated by the hot blanket method and showing the holes due to settlement of foundation, heaving by frost action, or wearing away, caused by lack of adherence to the base, the earlier such defects are repaired the longer will be the life of the treatment. After 2 years' service of a treatment of this nature, during which time not more than one-third of the surface should be in need of repairs, it becomes a question of the advisability of re-treating the entire surface or of tearing it up and resurfacing with a more permanent pavement. For the repair of minor holes, a well organized gang can readily keep a stretch of road in perfect condition at a moderate outlay for plant and materials. Such a repair outfit should have a portable kettle, for heating the bituminous material, and a compartment for keeping dry the grits or screenings. The hole to be repaired should be swept clean and painted with hot bituminous material of the same consistency as that used in the original treatment, and then filled with grits or screenings to the level of the surrounding surface. This should then be grouted with bituminous material and covered with a light layer of stone chips or gravel. Where practicable, the patches should be rolled, and should be carefully watched to see that the traffic does not tear them out before they properly set. Another method of repairing such bituminous surfaces successfully is by mixing the aggregate and bituminous material away from the site of the road and depositing them in the hole to be repaired. This should be tamped, sanded, and thrown open to traffic. The writer believes this method leads to more uniform results.

Bituminous Pavements.—The maintenance of the older types of bituminous pavements has been sufficiently studied to need no elabora-

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tion here. This refers to bituminous concrete, sheet-asphalt, asphalt block, and rock asphalt pavements. The more recent bituminous macadam pavements constructed under the penetration method in many cases show the need of immediate repairs. A study of the causes responsible for such failure should be made with a view of obviating these defects in the original construction. The writer, however, does not at all agree that the penetration method is the total failure that some authorities would have us believe, but is convinced, on the contrary, that for certain kinds of traffic this type of pavement is good enough. This conviction is based on the features of low first cost, length of service, ease of repair, and moderate charge for plant rental with which to apply the bituminous materials properly. Penetration pavements have now been down for a period extending through five summers and winters, have had no repairs spent on them, and are still in perfect condition. Where failures have occurred they can generally be traced to some defect in the original construction. The main causes of such failures are: (1) improper foundation; (2) improper sizes of mineral aggregate; (3) sealing of top course before applying bituminous material; (4) bituminous material of wrong consistency; and (5) uneven application of the materials. The various ways that failures occur in work of this nature are shown on the surface by: (*a*) wavy condition, showing either an unequal distribution of bituminous material or lack of bond to the underlying stone; (*b*) depressions in the surface, due to settlement of the foundation; (*c*) raveling, due to the bituminous material not adhering to the stone; (*d*) surplus of bituminous material on the surface, due to the difficulty of penetrating the stone, combined with improper consistency. For patching holes in a penetration road it is best to clean it thoroughly, scarify the surface, and add sufficient stone to bring the top course to the level of the surrounding road. The heated bituminous material is then applied, and stone of the next smaller size is rammed into the interstices. Generally, only one application is required, and care should be taken that the patch is at the same level as the surface of the road. Where there is an extreme wavy condition, it would probably be found more advisable to scarify the entire surface, adding sufficient stone, if necessary, and treating it with a light application of bituminous material followed with screenings and thorough rolling. Where raveling has taken place, a blanket treatment of bituminous material, applied under pressure, will often seal the road, without the necessity of scarifying.

Bituminous concrete pavements laid cold (including bituminous mortar) are ordinarily maintained with material prepared at some central point and shipped to its destination. This may be used to repair holes or to spread over the surface to form a new roadway.

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Equipment.—For the application of bituminous material, by far the most successful method is by the pressure distributor. This may be horse-drawn, operating a small pump by a sprocket chain, or the pump may be driven by a gas engine in the rear of the wagon; but the most economical is the auto-driven sprayer. The speed can be regulated to spread any quantity required in a uniform spray, and under uniform pressure. For the blanket treatment, there has recently been placed on the market a machine designed to apply heated materials under pressure. This is considered the most advanced and improved method of applying heated material, and its use should be encouraged wherever possible. The outfit consists of a double-shell, asbestos-lined tank, holding from 750 to 1 000 gal., and containing 1½-in. heater pipes. A boiler, equipped to burn either coal or fuel oil, is carried on the rear. This furnishes steam to heat the bituminous material in the tank and to operate an air compressor. The bituminous material is forced through the outlet nozzles under the pressure desired, the quantity being gauged by the speed of the truck and the size of the outlets. This machine can apply the heavier material used in the construction and maintenance of bituminous pavements under the penetration method, and can readily apply the seal coat on bituminous concrete pavements. Where long stretches of road are in need of repairs, the auto-truck sprayer will be found the most economical, and where communities do not care to invest in the costly machines of this nature their services can be contracted for, on a gallon or yardage basis, from the companies operating them.

Indirectly related to the question of maintenance is the uniformity of the bituminous material to be used. There have been discussions at various times relating to the physical and chemical requirements of bituminous materials, and specifications, designed to limit and control their manufacture, have been drawn. Speaking only in reference to petroleum asphalts, with which the writer is more familiar, a few of the features governing the manufacture of the products under given specifications will be cited as illustrating some of the difficulties attending its preparation. In the first place, for a given grade of crude oil, a uniform product should result from the manufacturers' plant. On the other hand, elaborate tests may be interesting, as a matter of record and to classify the asphalt used in any given piece of construction, but when the manufacturer is asked to meet a variety of tests, necessitating a change in his process, a hardship is imposed on him, with no betterment of the resulting product.

The manufacturer who uses stills in the preparation of his material is governed largely by three fundamental conditions: (*a*) the character of the base material he is refining; (*b*) the temperature of distillation; and (*c*) the time of distillation. For a given crude supply, which may be considered satisfactory for the preparation of asphalt, it will be

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seen that the resulting product is dependent largely on the temperature and the time of distillation. In many cases any change in one to meet a given test will affect seriously more important and necessary tests. As an illustration may be mentioned the fixed carbon requirement in certain specifications. It is agreed, on excellent authority, that the fixed carbon in an asphalt is the product found by dividing the fixed carbon in the crude material by the yield. Therefore a blanket clause in specifications limiting the percentage of fixed carbon, without taking into consideration the crude material which is being run, imposes a condition which the manufacturer is powerless to meet. He cannot control the minimum of fixed carbon, which is governed entirely by the yield and percentage in the crude, and a certain leeway should be given him for the maximum, to allow for errors in the test, imperfections in the process, and causes over which he has no control. Temperatures may control this maximum; but, in making a product for the highest class of paving work, there is no incentive or desire on the part of the manufacturer to injure the product in any way possible, and he would have no object in running at a temperature other than that which will produce the best material.

Engineers are prone to adopt technical specifications for materials used by other communities without investigating their merits as applied to their own work. Although this may be taken as a compliment to the originators of such specifications, in many cases it is done through a lack of knowledge of the materials to be used, whereas the unbiased judgment of those qualified to speak, with an eye to the considerations of cost and durability, will aid many engineers in the intelligent preparation of specifications.

The opinions of engineers assembled to discuss almost any form of construction and tests on most grades of materials will represent the most up-to-date and best thought, and engineers cannot go far astray in adopting such specifications, provided they have been tried and in actual use. Therefore national societies have a duty to perform in perfecting specifications for paving materials which are to be used generally throughout the country, and care should be taken that such specifications contain only those items which will insure a satisfactory product for the work to be done and not impose conditions impossible to fulfill with the materials available.

Mr. Boorman.

T. Hugh Boorman, Esq.—In the discussion 2 years ago the speaker stated that Nelson P. Lewis, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment of the City of New York, had considered that rock-asphalt pavements were probably the best that had been laid. George W. Tillson, M. Am. Soc. C. E., objected to it because he considered the maintenance charges on it excessive. The speaker, therefore, begs to state that rock-asphalt comprimé has been

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laid in the United States in New York, Brooklyn, Long Island City, and Rochester, in New York State; Elizabeth and Perth Amboy, in New Jersey; Boston, Mass.; New Haven, Conn.; Philadelphia, Pa.; and New Orleans, La.

The first comprimé work was laid in Union Square, New York, in 1872. In August, 1897, 112th Street, from Fifth to Lenox Avenues was laid, and the city records show that in 1912 the cost of repairs on this street was only 6 cents per sq. yd.; and 101st Street, from Lexington Avenue to Park Avenue, paved in July, 1896, with Mons and Sicilian rock, bears no cost for repairs from 1910 to 1913. Such durability seems to be unparalleled in the history of street construction. On Dyckman Street, from Kingsbridge Road to the tracks of the New York Central and Hudson River Railroad, 8 000 sq. yd. of Seyssel and Sicilian rock asphalt were laid in the late fall of 1897, and the cost of maintenance for the 4 years, 1910 to 1913, inclusive, was 3 cents per sq. yd., or less than 1 cent per sq. yd. per year. In 1901, between Park and Lexington Avenues, 35th and 36th Streets were paved with Seyssel and Sicilian rock asphalt; on 36th Street no repairs were necessary for the 4 years, 1910-13. In the same year 13th Street, from Second to Third Avenues, was paved with Sicilian and Mons rock on an old stone foundation, and is charged with 14 cents per sq. yd. for 4 years' maintenance. In 1897, 106th Street, from Broadway to Riverside Drive was paved with Sicilian and Mons rock, and shows cost for repairs of 17 cents per sq. yd. for 4 years, a trifle more than 4 cents per year.

To come to more recent times, it was decided in 1912 by the Department of Highways, under the advice of E. P. Goodrich, M. Am. Soc. C. E., Consulting Engineer to the President of the Borough of Manhattan, to lay a series of test pavements on Second Avenue, from Houston to 23d Streets. Rock asphalt comprimé, rock-asphalt blocks, and Sicilian rock comprimé were laid between 19th and 21st Streets, and on inspection, in January, 1914, these two blocks were found to be in good condition, the only imperfection being in the work adjoining the car tracks. This is not to be wondered at, as all rock asphalt experts, such as Malo, Delano, Walsh of Amsterdam, Bassett of London, and almost all engineers in Europe have decided that granite or other blocks should be laid longitudinally beside car tracks. The speaker recommends scoria blocks. The most noticeable of all the experimental pavements on Second Avenue is that of rock asphalt compressed blocks, between 9th and 11th Streets. These blocks were manufactured, at the Staten Island Works of the Sicilian Asphalt Paving Company, in an ordinary German brick machine, and did not receive the perfect compression obtained by the Val de Travers Asphalte Paving Company in their works at Marseilles, Seyssel, Cairo, Egypt, and other places. However, the rock being pure crude rock asphalt powder before being

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pressed, simply spread under heavy traffic, and became virtually a monolithic sheet of rock asphalt, and it affords to-day as perfect a specimen of rock asphalt pavement as can be seen in any country.

Mr. Delano, on the occasion of his visit to New York in November, 1913, stated that the City of Paris had recently signed contracts for replacing wood blocks, stone sets, and macadam with 700 000 sq. m. of rock asphalt pavement, to be laid during the next 5 years. Attention is also called to the fact that, some years ago, Thomas Street, from Broadway to Church Street, and Trimble Place, from Thomas to Duane Streets, both private streets owned by fifty associates, were paved with Neuchatel Rock Asphalte Pavés and coated with an asphalt rubber surface coat, only $\frac{3}{8}$ in. thick, which has stood the heavy traffic of those busy thoroughfares for several years.

The speaker can prove that rock asphalt is the most valuable pavement in the United States. This is stated especially because so many engineers doubt that any European rock asphalt, such as laid in London, Paris, and Berlin, has ever been laid in the United States. The speaker is sorry that, owing to the short time allotted to him, he will not have the opportunity to say a little about its history in America.

The reason this subject of the practicability of the use of European Rock Asphalte is so much more important than when the discussion before the Society was held in 1912, is that the tariff on asphalt has been removed, and Rock Asphalte powder can now be purchased at a cost of $3 per ton less than in 1913, making a difference of 25 cents per sq. yd. in the cost of the surface. Municipal plants can be supplied with the material, all ready for heating and compressing on the street, and no additional outlay for machinery is necessary. Both the Borough of Manhattan, which is now completing its new municipal asphalt plant, and the City of Philadelphia are considering the advisability of doing their own repairs to rock asphalt streets.

Mr. Sharples.

PHILIP P. SHARPLES, ESQ.*—One or two points brought up in the discussion were not entered into very fully. For a number of years the speaker has been very much interested in studying the effect of oil on tar-bound macadam pavements. In considering the subject, country road practice and city or town practice must be sharply differentiated.

In building country roads, such as those described by Mr. Farrington, it has been the practice in many cases to construct a tar-bound macadam road, and coat it with one of the heavy asphaltic oils or light asphalts put on hot. This practice has given excellent results in many cases, and has been established as a standard specification in some States, notably in Connecticut. Some very good work has been done in this way.

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Mr. Sharples.

In carrying out work under a specification of this kind, it is very important that the grade of oil or oil asphalt be so heavy that it will not penetrate the tar-bound macadam, but will stay on top as a blanket layer. An oil light enough to penetrate the tar-bound road is sure to give bad results. The speaker knows of a number of roads of this type, which were well built and then treated with a light oil, while still presenting a porous top. They disintegrated completely within a year when subjected to heavy horse-drawn traffic, although neighboring sections sealed with tar products remained in good condition. If, however, a proper grade of asphaltic oil is used, this disintegration will not occur.

Under city or town traffic conditions, the tendency of an oil-asphalt top over a tar-bound macadam is to mush up and become soft under horse-drawn traffic in wet weather, due to the emulsifying effect of the water and organic material which collects on the asphalt top. When these conditions are to be provided for, it is important, so far as present experience indicates, to use only tar seal coats on top of the tar-bound macadam. The refined tars used for seal coats are preferably somewhat softer than the tar used in the tar-bound macadam, but the subsequent treatment and care of the road modifies somewhat the character of the seal coat required.

For maintenance within city and town limits, where the horse-drawn traffic is considerable, it has been found that a specially prepared tar, which can be applied cold, gives much better results than the heavier tar materials put on hot.

There are records of streets, in the vicinity of Boston, which carry more than 4 000 vehicles per day, which have been kept in good shape since 1908 with an annual maintenance cost of about 3 or 4 cents per sq. yd., with prepared tar applied cold.

Consider the light dust-laying oils and their effect on tar-bound macadam. It is a problem which affects city and town conditions. In country practice, the conditions do not arise where an oil of this type is necessary. On city and town streets, a troublesome dust often collects, which the ordinary seal coat of tar or heavy asphalt does not keep down. In applying a light oil over a tar-bound macadam to keep down this superficial dust, it is essential to comply with certain conditions if the tar-bound macadam is not to be ruined. It is the utmost folly to put a light oil on a tar-bound macadam; if there is the least chance for the oil to penetrate the macadam, the results are sure to be disastrous. The tar-bound macadam will be disintegrated. A light oil has two effects on the tar-bound macadam. It has a chemical effect on the tar which causes its disintegration and makes it lose its binding force. It also has a physical effect on the macadam structure. The oil is a lubricant; and if there is any stone loose enough to move in the tar macadam, the light oil accelerates the movement,

Mr. Sharples.

and a hole is sure to develop. This dual action of the light oil can be observed in many large cities at the present time, and many streets may be seen which have been built with tar-bound macadam, the life of which has been gradually shortened by the use of light oil.

The effects of the light oil can be minimized if the tar-bound macadam is thoroughly sealed, either through an excess of bituminous material in the first place, or by subsequent applications of refined tar. Certain precautions must be taken also in applying the oil. An excess of oil should never be used, and it should be applied to the surface only in sufficient quantity to take up the loose dust which is found there. A very small fraction of a gallon is sufficient. If more than this is applied, the oil gradually sinks into the tar-bound macadam and in the end will rot it.

Wherever a light oil treatment is used on a city or town street for dust-laying purposes, it is quite essential to revivify the surface of a tar-bound macadam from time to time by the application of a tar seal coat. For this purpose nothing seems to be better than a prepared tar of such consistency that it can be applied cold. Only a very small quantity is required, usually not more than ¼ gal. per sq. yd. per annum. If these applications are put on intelligently, and if patching is intelligently attended to, a tar-bound macadam will give good service, even in the face of light oil applications.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

BY L. J. LE CONTE, M. AM. SOC. C. E.

Mr. Le Conte.

L. J. LE CONTE, M. AM. SOC. C. E. (by letter).—The writer is much pleased with the results of the investigation given by the author. In May, 1901, he wrote a paper on this interesting subject for the American Water Works Association. This paper was intended only as a rough general guide to aid the judgment of engineers who were in trouble. Since that time he has been called on repeatedly to explain. Accordingly, he gives the following specific case:

Let x = average rainfall for the 3 dryest years on record.

The average run-off $= \frac{x}{2}$

The average catch $= \frac{x}{2} \times \frac{1}{4} = \frac{x}{8}$.

Assume x $= 40$ in.

Then average catch $= \frac{40}{8} = 5$ in. per annum.

$5 \times 17\,400\,000$ gal. $= 87\,000\,000$ gal. per sq. mile; continued for 3 years $= 260\,000\,000$ gal. per sq. mile.

Where local experience indicates that it takes 2.5 sq. miles of water-shed to furnish 1 000 000 gal. per day in a long dry spell, then $260 \times 2.5 = 650\,000\,000$ gal. storage, which is necessary to insure a supply of 1 000 000 gal. per day in a long dry spell.

* Continued from February, 1914, *Proceedings*.

Mr. Le Conte.

Referring to Fig. 32, the writer is pleased to note a practical corroboration of this result. The Croton storage is given as 101 300 000 000 gal., and the water-shed as 360 sq. miles. This makes a storage of 290 000 000 gal. per sq. mile, for New York. The Spring Valley Reservoirs, at San Francisco, Cal., have 30 000 000 000 gal. storage for 30 sq. miles of water-shed, or, 1 000 000 000 gal. storage per sq. mile.

Then, $\frac{1\,000}{290} = 3.45$ times as much storage as is required in New York. Why? Simply because at San Francisco it takes 5 sq. miles. of water-shed to furnish 1 000 000 gal. per day, whereas in New York it only requires 1.5 sq. miles to deliver that quantity. In the New York case then, $260 \times 1.5 = 390\,000\,000$ gal. storage, and in the San Francisco case $260 \times 5. = 1\,300\,000\,000$ gal. storage.

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REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.

Discussion.*

BY MESSRS. A. W. BUEL, AND EDWARD WEGMANN.

Mr. Buel.

A. W. BUEL, M. AM. SOC. C. E. (by letter).—The author assumes the ratio of the coefficient of elasticity of steel and concrete at 10, which would seem to be low enough for any purpose, and sufficiently safe. Then, in the formula for the value of T, if the writer understands it correctly, the author seems to use a ratio of 9, making the tension in the steel, for 290 lb. per sq. in. in the concrete, only 2 610, instead of 2 900.

It would be interesting to know what the comparative cost would have been if, instead of proportioning the thickness of the concrete on the theory used by the author that vertical cracks would not develop if the tension be limited to 290 lb. per sq. in., the concrete had been made with a uniform and minimum thickness from top to bottom and with a separate water-proof lining inside. Unless there is a considerable saving in cost, the separate water-proof lining would appear to have the advantage of the other method, on the ground of many successful precedents.

Mr. Wegmann.

EDWARD WEGMANN, M. AM. SOC. C. E.—Although not directly a discussion of the paper under consideration which the speaker has been asked to discuss, it may be interesting to describe the recent partial failure of the reinforced concrete Ambursen dam, built in 1912 and 1913 across Stony River in West Virginia, for the West Virginia Pulp and Paper Company.

In the fall of 1911, this company engaged the speaker's services as Consulting Engineer to examine the site selected for its dam, report whether it was suitable for this purpose, and recommend the

*Continued from February, 1914, *Proceedings*.

Mr. Wegmann.

type of dam that should be constructed. The dam was to be built on top of a mountain at the end of a log railroad. Owing to the great depth to rock, the cost of a masonry dam was prohibitory. There was no suitable material available for an earth dam, and therefore the speaker recommended that a hollow dam of reinforced concrete be built.

After a number of test pits had been dug and some borings had been made, four construction companies were invited to submit plans for the dam and bids for its construction. Before doing so, each of these companies sent a representative to inspect the site of the dam and the test pits. The plans submitted have been fully described.* Each construction company bid a lump sum for building the dam with a cut-off wall down to a certain assumed probable depth. Below this depth the cut-off wall was to be paid for by the cubic yard.

The dam was to be about 1 000 ft. long and was to have a maximum height of about 50 ft. above the surface. The lowest bid was $143 000, received from Mr. F. G. Webber. The bid of the Ambursen Hydraulic Construction Company, of Boston, was $189 000. In view of the fact that the latter company was the only one of the four bidders which had built a large number of reinforced concrete dams, the speaker recommended that the Ambursen Company be engaged to make the plans for the proposed dam. This was done. The contract for building the dam was awarded to Mr. Webber. To insure that the work would be properly done, the Paper Company engaged one of the engineers of the Ambursen Company to be constantly on the ground during the construction.

Mr. Webber began work at the west end of the dam and built about half the structure. The work was then taken away from him, as the progress he made was not satisfactory, and the dam was completed by the Ambursen Company at cost plus a certain percentage for profit.†

It appears that for about 200 ft. from the west end of the dam, the cut-off wall was founded on hard clay, and was made only 5 ft. deep below the floor of the dam. This shallow cut-off wall was built to where the water was from 30 to 35 ft. deep. For the remaining length of the dam, the cut-off wall was founded on rock. This part of the dam has remained intact.

The failure was caused by water finding its way under the cut-off wall, through a permeable, narrow seam of stone, sand, etc., that occurred in the clay formation. If the cut-off wall had been carried down a foot or two deeper, it would have intercepted this seam.

The water that passed through the seam showed itself in the weepers of the floor three or four days before the failure occurred. The

* *Engineering News*, September 5th, 1912.

† The manner in which the dam was built is described in *Engineering News*, of January 22d, 1914, and in the same issue there is an account of the failure of about 75 lin. ft. of the dam, three or four months after the reservoir was filled.

watchman, left in charge of the dam, did not appreciate the danger this leakage indicated. It was only a day before the failure, that it occurred to him to inform the Superintendent of the Paper Mill at Luke, Md., about the leakage, which was steadily increasing in volume. Mr. Wegmann.

At that time a blizzard was raging and the temperature was 10° below zero. When the Superintendent arrived, the following day, the dam had been undermined and about 75 lin. ft. of the structure was destroyed.

The accounts of this failure which appeared in the daily newspapers were greatly exaggerated. The reservoir stored only about 800 000 000 gal. From the dam to the Potomac River, Stony River flows through a wilderness, the only building being a trapper's hut. The damage done, exclusive of the destruction of part of the dam, was very slight.

The failure was evidently due to the shallowness of the cut-off wall. In asking for lump bids for building the dam, the engineer of the Paper Company and the speaker had to assume certain depths to which the cut-off wall would probably be built. This depth was made 7 ft. below the top of the floor of the dam at the ends of the structure, 10 ft. where the depth of water was from 20 to 25 ft., and the wall was to be carried down to rock in the center of the valley. At each end of the dam sheet-piling was shown. The depth to which the cut-off wall was to be built had to be determined, of course, during the construction.

As actually built, the core-wall was founded on rock for about three-quarters of the length of the dam, and for the remaining length it was made only 5 ft. deep below the bottom of the floor of the dam. No sheet-piling was used.

The speaker's connection with the work ceased when the contract for building the dam was awarded, which was before the plans prepared by the Ambursen Hydraulic Construction Company had been received, and he had nothing to do with the construction.

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STEEL STRESSES IN FLAT SLABS.

Discussion.*

BY MESSRS. EDWARD GODFREY, H. E. ECKLES, AND SANFORD E. THOMPSON.

Mr. Godfrey.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—The flat slab has repeatedly been brought before the Engineering Profession for consideration and adoption, on the plea that it is efficient and economical. Before adopting it, however, the Profession has a right to examine its credentials. The theory of the flat slab must pass muster; and tests on it must be consistent and rightly interpreted. Some things in reinforced concrete design have been adopted, which have not met these conditions. They were adopted in ignorance and have been the cause of unnumbered wrecks. One of these is the stirrup or short shear member, another is the rodded column with no lateral reinforcement; but these will be rooted out in time. That they ever received the sanction of the Profession is a disgrace. It will not do to add to the number of such ill-considered standards by the adoption of any other untried and illogical method of design.

The theory of flat slabs, as given by most writers, would tend to place it in the innocuous class, for the great thickness of slab demanded would put it "out of the running"; but the commercial designers succeed in scaling down the bending moments to a point which independent practising engineers fear to attempt.

The flat slab theory premises a slab of indefinite extent, supported on an indefinite number of columns, the entire slab being uniformly loaded. The term, cantilever flat slab, so often used, conveys the idea that over the column heads the slab is a cantilever. The premises and the several assumptions are faulty. The slab is not of indefinite extent.

* This discussion (of the paper by H. T. Eddy, Esq., published in January, 1914, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Godfrey.

It must end at the outer row of columns or a short distance beyond them, and sometimes there are only two or three bays. Sometimes there is a girder or wall at the outer edge and sometimes there is not. Manifestly, when the outer row of columns is reached, there must be—in order to satisfy the theory—something to take the place of the continuous slab that will supply its equivalent. A girder or wall on the line of the columns does not perform this office, as a girder cannot hold a slab in a fixed horizontal position as though it were a cantilever. A continuation of the slab beyond the column line cannot supply the condition necessary, unless it is uniformly loaded with just the proper balancing load. This uniform load, however, would of necessity include some live load, and there is no law compelling an owner to dispose the load on his floor in any given way.

The idea that the slab is a cantilever over the column means one of two things. Either the load on the slab must be properly balanced, or the columns must be subject to a bending moment due to live load to one side of the column line. To assume that the live loads are always properly balanced over the columns is to restrict the owner in the free use of his structure and to demand something unthinkable. To make the columns carry the cantilever load of the slab is to subject them to stresses for which they are never calculated.

On page 67* the author says:

"The stress at 11 in the wall panel, *C*, was larger, and amounted to 20 500 lb., which probably was due to the cantilever action at the wall being less than that exerted by the usual column heads."

Here is a confession that the column heads, which means, of necessity, the columns, are depended on to take cantilever stresses from the floor.

It may be urged that the stiffness of the slab will make it capable of taking at least a part of the bending moment beyond the column line. It cannot possibly take all of it without either infinite stiffness or articulation in the column. Any theory is wrong that does not follow to an ultimate and adequate support or resistance the loads or strains assumed on part of a structure. A slab theory that ignores the effect of the slab on the columns, and leaves the reader to infer that these columns are centrally loaded, is wrong.

Another part of the theory that is wrong is the assumption that Poisson's ratio has any application to the steel reinforcement. The author's theory for the mushroom slab assumes that the steel reinforcement is equivalent to a continuous flat sheet of steel. This is contrary to fact and reason. A set of cross-rods does not in any sense fulfill the conditions that make it equivalent to a single sheet of steel.

* *Proceedings*, Am. Soc. C. E., January, 1914.

Mr. Godfrey

It is freely admitted that when a sheet of steel is pulled in one direction it contracts in the other direction. As to whether this contraction is equivalent to a compressive force that would give the same amount of shortening is another question. If this is applied to two sets of transverse rods, an entirely different condition exists. The only medium between the sets of rods is the concrete. In a previous discussion* the author has stated that for the mushroom type Poisson's ratio is five-tenths. For the type that he delights to honor, then, here is what this statement means. A 1-in. square rod stressed to 16 000 lb. is pulled crosswise by the concrete 8 000 lb. for each inch of its length. The concrete might have an adhesion one one-hundredth of this amount. Where does the other ninety-nine-hundredths come from?

In asking the Profession to accept this mushroom theory, the author is asking us to consider the laws of matter suspended for this particular type, or else that there are special laws that apply to this special combination of materials. The theory of flat slabs is hard for any disinterested engineer to accept; let us look at the tests.

The author finds agreement between his theory and the result of tests. According to his statements, deflections and extensions in the steel would seem to bear out his theoretical deductions. This apparent agreement is explained by two things, one is tension in the concrete, and the other is large columns that are only partly loaded by the tests and, therefore, can lend some of their unused strength toward resisting cantilever action.

There can be no doubt that tension in the concrete plays a large part in resisting bending moments, and consequently carrying the loads. This is true of other forms of reinforced concrete, as well. In a simple beam or slab a very large part of the bending moment is resisted by tension in the concrete; but the accepted theory ignores this tensile strength entirely, and wisely so. This is the reason that deflection calculations for reinforced concrete beams, based on no tension in the concrete, are worthless.

In a flat slab, the tensile strength of the concrete has vastly more influence in supporting loads or resisting bending moments than in a simple beam, because of the fact that the tension is in all directions. Deflection is diminished and steel stresses are reduced by reason of this tensile strength of the concrete, and any theory that tries to explain these results through the supposed influence of Poisson's ratio is false. It cannot be gainsaid that, if a flat slab were cracked along certain lines, the steel stress and the deflection would be found to be very much more than they are in a whole slab, just exactly as the same would be true in any reinforced concrete beam or slab.

Tests on a flat slab, that are manifestly very greatly influenced by the tensile strength of the concrete are falsely interpreted, when the

* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1368.

Mr. Godfrey.

effect of that tensile strength is totally ignored, and, instead, Poisson's ratio (a thing that applies only to "any piece of material which is subjected to stress, and is of such shape that more than one of its dimensions is considerable,"* hence not to a rod) is brought in to explain the low stress in the steel.

If the commercial investigators frankly admitted that the tensile strength of the concrete is their mainstay, users of the flat slab would adopt it with their eyes open, if they had the temerity to use it.

The author has published† a test on a mushroom slab which purported to show the great strength of this style of construction and its agreement with theory. This slab, though only 18 ft. square, was supported on four comparatively enormous and very short reinforced concrete columns. It was loaded with balancing loads, that is, the first and second increments of the load were partly within and partly without the square enclosing the columns, which was 12 ft. each way.

Of course, such columns would have large influence in carrying cantilever loads, because of their 18 in. of width and their small direct load, but this is totally ignored in the author's theory, and all the strength is attributed to the virtue of the system. A building could not be commercially designed with columns about half as thick as their height and a safe load of less than 100 lb. per sq. in. This test is so far from representing anything in practice that it has not even a theoretic interest, especially when the theory absolutely ignores the greatest factors in supporting the loads.

Furthermore, balanced loads on this test slab tell nothing whatever of what the slab would do if the loading were confined within the square enclosing the columns.

The dishing effect of a flat slab is often referred to as explaining the strength exhibited in tests and there is no doubt that for an interior loaded bay, with the surrounding bays idle, the dishing has a large influence. The writer has shown, by tests,‡ that though dishing in an interior panel greatly increases the strength of that panel, a row of loaded panels will not show this dishing, and will not have this large strength. The tests which have been made on flat slabs in buildings have been on one or more interior panels (including in some cases exterior panels supported on walls or girders) but never, so far as the writer can learn, on a complete row of panels across a building. Under such loading, the slab would tend to take a cylindrical shape instead of being dished, and the concrete and the steel will be under very much greater stress.

A flat slab on rows of columns is no better conditioned than the same slab supported on parallel lines of girders. No commercially

* As defined by Dr. Eddy in *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1364.

† *Engineering News*, March 27th, 1913.

‡ *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1358; and *Engineering and Contracting*, July 2d, 1913.

Mr. Godfrey

designed flat slab will stand up under this criterion, that is, considering the slab supported on two lines of girders and figuring its bending moment as an ordinary slab subject to the common laws of Nature to which other than "flat slabs" are amenable. The writer has made these statements a number of times; they have never been controverted. This is the case of the flat slab in a nutshell. It stands or falls on the criterion just mentioned. In one notable case it fell with disastrous results.

The most serious aspect of this reliance on tension in concrete is in the case of rolling or jarring loads, as for a freight terminal or a viaduct. Repeated jarring of the load will in time crack the concrete. Then the steel will get its full load, which is several times that for which it is calculated, and trouble may be expected.

Mr. Eckles.

H. E. ECKLES, M. AM. SOC. C. E. (by letter).—The writer is glad to note the author's attempt to formulate conclusions in a more practical way than has characterized some of his past efforts. The assumption of a value for Poisson's ratio for steel and concrete in combination at four or five times its probable value, as shown by the published results of experiments with these materials in combination, and one-third larger than the sum of the ratios for the materials when not in combination, and used by the author in one of his published works in the derivation of formulas similar to those presented in this paper, accounts for some of the discrepancies between former theories and conclusions based on facts. The variations in flat-slab design and the growing importance of this type of construction are so great that attempts to formulate conclusions should be made with care, and with a view to general applicability rather than in support of a particular type of construction.

Much has been written regarding the action of flat plates under loads, and the writer believes that considerable of what has been put forth as a proper basis of design by advocates of reinforced concrete flat-slab construction is erroneous, and that the formulas proposed by the author in this paper give resultant stresses in the steel much less than would be developed under working conditions and proper test loading.

It should be noted that the results found by using the proposed formulas are compared with the observed stresses in the steel in the slab of Panel D when there was little load on the panels adjacent to it. In the adjacent Panel A the equivalent uniform load was about 260 lb. per sq. ft. of floor; in Panel C it was about 30 lb. per sq. ft.; and in the two other adjacent panels there was no load whatever. Our knowledge of this construction warrants us in believing that the effect of the surrounding floor in supporting the load of a single panel is quite large.

Mr. Eckles.

The writer recently conducted a test of a floor of the "mushroom" type of construction, having panels of the same dimensions and with a slab 7 in. thick, reinforced, however, with a larger percentage than usual of slab steel. The design load was 200 lb. and the superimposed test load was 400 lb. per sq. ft. of floor. The results of this test showed a remarkably large effect on the deflection at the middle of the panel, when parts of the adjacent panels were loaded with a full test load along two sides.

The adjoining panels were of the same dimensions and reinforcement as the one loaded. The panel tested was at the corner of the building, supported on two sides by wall beams, and corresponding to that of the corner panel shown in Fig. 1, having Columns 9, 10, 11, and 48 at the four corners.

In the first stage of the test, the panel only was loaded. Care was taken to leave clear passages so as to prevent any arch action.

The second stage consisted in loading areas 4 ft. 6 in. wide adjacent to two sides of the loaded panel, for the full length of the side of the panel, with the same uniform load as that on the panel. This placed a full test load practically over the full width of the belt of bars extending from column to column and parallel to the sides of the panel. After the placing of this additional load, the measured deflection at the center of the panel was exactly twice that found when only the full area of the panel was loaded. The measurements were made very carefully by two direct methods, to avoid any error, and a period of 40 hours was allowed to elapse between the time of the loading of the panel and the loading of the adjacent strips of floor. It seems probable that the difference in the deflections would have been somewhat larger had the width of the adjacent loaded strips been greater.

This is in harmony with the results of tests given in papers presented* some months ago, in which it was shown that loads are distributed laterally for a distance beyond the boundaries of the loaded area, in some cases amounting to more than one-half the span of the slab. Similar conclusions have been reached as a result of other tests. The method of arranging the reinforcement in this type of construction, shown in Fig. 1, with the belts of bars overlapping each other near the columns, would seem to indicate clearly that, for a single loaded panel, where the side belts are only partly loaded, the effective span of the diagonal belts would be decreased or largely modified. This effect would be enhanced still further by the fact that the side belts are of shorter spans and consequently subject to less deflection. This is precisely the effect of the "drop," used in the Larkin Building, and would seem to account for the approximate agreement of the results

*At a meeting of the American Society for Testing Materials.

of the calculations by the author's use of his formula, with those of the Larkin test, where the load extended over several panels. Mr. Eckles.

It has been pretty clearly established, by the numerous tests made, that the points of contraflexure are fairly close to that which the common theories of flexure would indicate, considering an elementary width of slab extending from column to column to act as a beam.

In the "mushroom" system this point is much nearer the column than in cases where the "drop" is used, and calculations made on this basis give results considerably in excess of those by the author's proposed formulas, based on partial test loads. The author has fallen into error through his disregard of the well-established laws of flexure, with the result that the conclusions in his paper are erroneous.

The advantages of the flat-slab type of construction are obvious. With the same quantity of material, the structure can be made approximately of the same strength as when the usual slab-and-beam type of construction is used, without excessive deflection, and with the added advantage, in many cases, of greater clearance, or a reduction of the cost by a less height of structure being required. With these facts in view, it is questionable practice to advocate a flat-slab type of construction designed under specifications inconsistent with what is recognized as good practice in the more common slab-and-beam type.

One of the disadvantages of the "mushroom" system without the "drop" is the lessened strength in shear of the slab around the column head. At the beginning of the paper the author has advocated the use of small percentages of reinforcement, though his reasons for this are not apparent. The quantity of reinforcement often used in the "mushroom" system is one-fourth of 1%, and, considering the possible effect of the overlapping belts, with this percentage the neutral axis of the slab would be approximately three-tenths of the effective depth from the compression face at points near the column. In a slab of 8 in. effective depth, the neutral axis would be about 2½ in. from the face of the slab.

The compression side of the slab is the only part of its cross-section which is effective in shear; the shear developed on this part of the slab around the top of the column is a measure of diagonal tension, and is of very high unit value. Where a "drop" is used, this unit shear is greatly reduced. With the same total quantity of reinforcement and a "drop" of half the thickness of the slab, the unit shear is reduced approximately one-fourth.

SANFORD E. THOMPSON, M. AM. SOC. C. E. (by letter).—Dr. Eddy fails to bring out the following important conclusions from the tests on the Northwestern Glass Company's Building: Mr. Thompson.

1. The reinforcement of the slab at the column head is entirely inadequate for the design load, so that the tensile stresses in

Mr. Thompson.

the steel and the compressive stresses in the concrete are excessive.

2. The gauge lines at the column head were not placed properly, so that the readings of the stresses do not represent the maximum stresses in the steel at the column head.
3. Tests made on single panels, or with the loading similar to those on the Northwestern Glass Company's Building, Loads 5 to 7, do not produce the largest stresses in the most important part of the structure, that is, at the column head.
4. The wall panels should be designed differently from the inside panels.

Stresses at the Column Head.—By the nature of the construction, the flat slab derives its strength from the rigidity of the column head. For this reason, it is absolutely necessary to design the slab so that the tensile and compressive stresses at the column head are within working limits. In analyzing the results from the tests, it is most important, therefore, to pay close attention to the stresses at the column head.

For the column head, the most unfavorable position of the loading is when all the panels around it are fully loaded. In the Northwestern Glass Company's Building, the only loading that caused the most unfavorable condition at the column head was when Load 3 was applied, and then the total load per panel was from 93 000 to 97 000 lb., though the panels were designed for a total live load of 102 400 lb. Even for this smaller load, however, the stresses in the steel due to the live load alone reached 22 000 lb. per sq. in. If to this is added the stress due to the dead load, the total stress for the design load is more than 50% greater than that usually allowed in reinforced concrete construction. The stress referred to is at Point 111 (Table 12). Stresses just as high undoubtedly would have been found elsewhere, if the other gauge lines had been properly placed over the edge of the head, as explained later. That this stress was not abnormal is indicated by the uniformity with which it increased under the load and then decreased when the load was removed.

Dr. Eddy attributes the high stress at Point 111 to the fact that:

"It was necessary to pry the ends of the rods upward forcibly and hold them in this position by blocks under their extremities, thus putting them under considerable initial bending stress."

As the gauge line, 111, was on a straight bar belonging to the diagonal band, it is difficult to see how lifting the steel in the bands, which were not attached to anything, puts any initial bending stresses on it. Even if initial stresses existed, they would not have affected the readings of the stresses, as extensometer measurements do not indicate the actual stresses in bars, but only the differ-

ence between the stresses at the initial reading and at the respective reading under load. Any initial stress, like stresses due to the dead load, is not included in this difference. Mr. Thompson.

Mr. Eddy states, further:

"At 111, Column 36, on the edge of the loaded area, the abnormal value of 22 400 lb. was reached under Load 3. Load 5 shows no such large increase over Load 3 of observed stress in the slab rods of the column heads in Panel D as was to be expected by practically doubling the loading. This fact is apparently inexplicable."

This statement is absolutely misleading. Point 111 is in Panel C, and this panel received its largest loading with Load 3. With Load 5, Panel C had a load of only 39 lb. per sq. ft., as may be seen by referring to the detail of loading in Fig. 6. It is entirely obvious, therefore, that the stress at this point ought to be less with Load 5 than with Load 3.

Faulty Location of Gauge Lines.—The location of the gauge lines over the column head is unfortunate. As is evident from Fig. 3, all gauge lines, except 103, 107, and 111, are inside the column head. The readings, therefore, on these gauge lines cannot represent in any way the actual maximum stresses in the reinforcement of the column head.

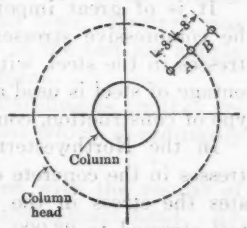

FIG. 20.

The writer has in his possession tests made by one of his associates on a similar construction, where readings were taken on gauge lines placed as shown by Fig. 20. The average results from this test gave at *A* a stress of 11 000 lb. per sq. in., and, at the same time, the stress at *B* was 24 000 lb. per sq. in. Similar results were obtained in the Worcester test, made under the direction of Mr. Brown. It is evident, therefore, that for gauge lines located like those used in the Northwestern Glass Company's Building, the results are much smaller than the maximum stresses.

This rapid decrease in stresses explains why, in Table 12, all those except 107, 108, and 111, were very low. The low stress in 103, which was placed outside the column head, is evidently caused by the fact that the gauge line was too far from the column head, and the bar was one of the outside ones of the diagonal band.

That Dr. Eddy realizes that the stresses in the rods inside the cap are much smaller than those in the same rods just after they leave the cap, is evident from the following:

"Although that part of the length of the rod which is inside the cap has its elongation prevented by the mass of the cap, the part outside must have its elongation correspondingly increased to compensate for this loss, and, on the whole, be equal to that of the rods beside it."

Mr. Thompson.

In analyzing the results from the test, however, he accepts the stresses inside the cap as the maximum stresses.

Stresses in the Concrete at the Column Head.—Dr. Eddy does not mention the stresses in the concrete at the column head. In the first part of his paper, he analyzes the relation between the stresses in the steel and in the concrete, and the influence of the percentage of steel on the stresses in the concrete. This analysis would lead one to believe that the stresses in the concrete, in the tests described later, are within working limits. This, however, is not the case. The case of "under reinforcement," mentioned by Dr. Eddy, exists within the panel. At the column head, on the other hand, the percentage of steel in the bands alone is four times that of the steel in each band. Wherever the bars overlap, the percentage is still larger. In designs similar to those discussed in the paper, the percentage of steel at the column head is between 1.5 and 2. For these percentages of steel, the stress in the concrete, corresponding to the working stress in the steel, must exceed the allowable working stress. This is evident from Table 1.

It is of great importance for the safety of the structure to keep the compressive stresses at the column head, as well as the tensile stresses in the steel, within working limits. Therefore, if a large percentage of steel is used at the column head, as is always the case in this type of construction, compression steel is indispensable.

In the Northwestern Glass Company's Building, an idea of the stresses in the concrete due to live load alone may be had if one calculates the stress in the concrete corresponding to about 1.3% of the steel stressed to 22 000 lb. With the dead load included, the stresses in the steel and the concrete would be at least 25% greater.

Loads 5 and 7 do not Produce the Worst Condition at the Column Head.—It is unfortunate from a scientific standpoint that the loading of all the panels was discontinued as soon as Point 111 received high stress. By further loading of all the panels, their true strength would have been obtained, but with the loadings as used, the results are simply misleading.

Loads 4*a* and 5, for instance, show Panels B and D loaded and Panels A and C unloaded. With this load, the stresses at the column, instead of being taken by the steel in one-fourth of the circumference of the column head, that is, by the steel tributary to that panel, were taken by the steel in one-half of the circumference. The correctness of this statement may be seen by reference to Table 12 where it is shown that although Panels A and C were not loaded, Points 103, 107, and 111, showed considerable stress. From this it is evident that the loading of Panels D and B was carried at the column head, not only by the steel belonging to Panels D and B, but also by the steel in Panels A and C. The stresses at the column head, therefore, with

this kind of loading, are much smaller than would have been the case had all spans carried loads of equal intensity. Mr. Thompson.

Other Tests on Mushroom Floors.—The student of this type of construction is struck by the paucity of reliable test data on the particular type of construction discussed by Dr. Eddy. Many tests are constantly referred to in printed literature, but on closer examination one is surprised to find that they bring out the stresses everywhere except in the most vulnerable part of the construction.

The test to destruction carried on by C. A. P. Turner, M. Am. Soc. C. E., or on his behalf, described in the engineering papers and more fully in Dr. Eddy's book, might have elucidated many mooted questions. The idea of a test panel with projections, if loaded properly, was a very good one, because the load on the projections could have been arranged so that it would have an effect almost similar to loads on the adjoining panels. The idea, however, was not carried out, and the slab was broken by loads placed in the center panel with comparatively little load on the projections. The question of the stresses in the most important part, that is, at the column head, therefore, was neglected, with the consequence that the results of the test are simply misleading to any one not well versed in the subject.

The test of the St. Paul Bread Company's Building is another instance in which everything was tested except that which is ordinarily the weakest part of the construction.

Formulas.—Dr. Eddy's formulas do not agree with the results of the tests in the most important place, *viz.*, at the column head. Although he calls them "theoretical formulas", the theoretical foundation is nullified by the many assumptions which do not agree with the actual conditions, so that the final results can lay no claim to being theoretically correct.

One of the assumptions for which no clear reason is given is that for the value of Poisson's ratio, K. As explained in Dr. Eddy's book, that ratio, for concrete alone, varies from 0.1 to 0.2. For steel alone this ratio is about 0.3. For a flat-slab construction, however, Dr. Eddy considers that neither of these two values is large enough, and he accepts a value of K equal to 0.5. To support this value, he has given a beautiful formula, but does not explain how two materials acting together can change their nature entirely, and how the top portion of a concrete slab, by merely being provided with steel at the bottom, can take on a lateral expansion equal to one-half the compressive deformation under stress. The writer does not see how this statement can be accepted by any one acquainted with the nature of the material, and yet on this assumption hangs the most vital element in the results.

The stresses for the steel in the diagonal and rectangular bands are calculated from formulas derived without regard to the size of the

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column head, and, according to Dr. Eddy, would apply to a construction with a column head equal to a sharp point and one of any size whatever. This assumption, on the face of it, is erroneous. The stresses in concrete at the column head are simply neglected.

In determining the stresses in steel over the column head, Dr. Eddy states: "The stresses in the middle rods of each belt, consequently, are increased abnormally for this reason just as it leaves the cap."

This statement, however, does not prevent him from continuing:

"Instead of attempting to determine this increase by some intricate investigation, it will be simply assumed that the stress at this point in the middle rod does not exceed that in the outside rod of the belt at a point opposite the center of the cap."

The available tests prove the correctness of the first statement. They show, however, that the stresses in the outer bars are much smaller than those in the middle bars. Dr. Eddy's assumption, therefore, is without foundation and, as a consequence, the results from the formulas do not agree with the tests. In the Northwestern Glass Company's Building, by using the actual live load on the panel, there is obtained by his formula a stress, $f_s = 15\,400$ lb. per sq. in., while the actual stress due to live load is at least 22 000 lb. per. sq. in.

In discussing the tests, Dr. Eddy uses, in his formulas for the total load in the panel, a much larger load than was actually on the panel. As a justification for this, he states that portions of the panels near the column heads were not loaded. He overlooks the fact, however, that there were two center aisles about 18 in. wide running across the center of the panel and thus taking away the load from the place where it would be most effective in producing bending moment. The use of the larger load is still more unjustifiable in calculating the stresses over the column head, because the stresses there are more affected by the amount of load in the panel than by their position. It appears that the same load uniformly distributed would not have caused larger stresses than was caused by the load placed as shown in Fig. 5.

In discussing the Larkin Building, Dr. Eddy makes the following statement:

"It would require a load of more than 1 200 lb. per sq. ft. on such a mushroom slab to produce a maximum unit stress on the slab rods as great as was caused in the Larkin slab by the test load of 738 lb. per sq. ft."

This, of course, is calculated according to his formula, which, as has been shown, does not agree within 45% with the actual results. This claim is only another expression of Dr. Eddy's faith in the miraculous properties of this particular type of reinforcement; but, to engineers familiar with the design of reinforced concrete, it seems at least open to discussion.

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As a conclusion, it appears, from a careful study of the tests, that a design similar to that of the Northwestern Glass Company's Building does not have the proper factor of safety required in reinforced concrete construction; that the stresses in the steel, as well as in the concrete, as is evident from Table 12, exceed those used in conservative designs; and that the formulas given by Dr. Eddy do not appear to agree with the results with the accuracy claimed by him; in fact, as stated before, the difference between the results from the formulas and the actual stresses reaches the large amount of at least 45 per cent.

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GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

BY MESSRS. S. HOWARD RIPPEY, S. C. HULSE, FRANK R. FISHER, H. L. COBURN, AND WILLIAM H. CUSHMAN.

Mr. Rippey.

S. HOWARD RIPPEY, M. AM. SOC. C. E. (by letter).—The deep interest in the subject of grouting dam foundations, as shown by numerous private inquiries from engineers in America and abroad concerning the Clackamas River experiments, renders it certain that this paper will be generally appreciated by the Profession, and it is to be hoped will develop a broad discussion of the general subject of dam foundation treatment. Although this subject is always of interest to those engaged in hydraulic work, its seriousness is from time to time given added significance by such unfortunate occurrences as the failures of the Austin and Stony River Dams, as well as of other less conspicuous structures, showing that perfectly well designed and constructed concrete dams of any type may be wrecked as a result of inadequate preparation of their foundations.

The detailed information given by Mr. Rands concerning the practical execution of the grouting programme at Estacada should be of value to others facing similar problems, but the writer fears that some of the conclusions reached by the author and the personal opinions expressed, may tend to indicate to others undue difficulties and limitations in the general applicability of the grouting method to dam problems, and to that extent discourage the use of a method which in many instances may represent the only economically practicable solution of a development project.

In order, therefore, that the results obtained at Estacada, and the conclusions drawn therefrom by Mr. Rands, may be properly appraised

*This discussion (of the paper by Harold A. Rands, Assoc. M. Am. Soc. C. E., published in January, 1914, *Proceedings*, and presented at the meeting of February 18th, 1914), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

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by others interested in the treatment of deficient foundations, it is proper to outline, as briefly as possible, sufficient of the history of that project to indicate the unfavorable character of the governing conditions under which the work was executed.

As indicated by Mr. Rands, the use of grout at Estacada was prescribed by the writer, following thorough study of the foundation requirements at the site of a proposed large reservoir and power dam on Clackamas River, some 5 or 6 miles above Estacada (known as the Upper "Clackamas" or "Second Clackamas" project). At this site a mass concrete dam about 150 ft. high was found to be desirable as part of the most efficient plan of power development and flow equalization, but the character of the foundations (similar to those found later at Estacada), was such that the writer felt that the construction of such a dam on them would not be in accordance with any known precedent, good judgment, or the dictates of good practice, unless the objections could be overcome. It thus became necessary to decide whether to modify the development scheme so as to avoid the necessity for a high dam (as, for instance, by a low dam and a long flume to create the desired head), or undertake to improve the character of the foundations so as to justify the execution of the more efficient and satisfactory plan of development.

At this juncture the writer decided that the use of cement grout, injected under pressure, offered a possible solution of the problem of foundation treatment, and probably represented the only practicable method of treating this remarkably heterogeneous formation, so as to justify the construction of a high masonry dam on it. Test-pit and core-drill investigations in progress indicated the absolute need for novel measures in order to render the foundations fit for the proposed construction, and the formations encountered were such as to make the writer believe that the grouting method could be applied successfully.

In order to acquire a more comprehensive idea of the possible range of geological conditions to be encountered than was feasible through the local borings in progress and by superficial examination of the canyon in this locality, the writer, fortunately, was able at this time to secure a very thorough geological report from Professor J. S. Diller, of the U. S. Geological Survey, who made a personal examination and study of the canyon for the purpose. In view of the questions raised by Mr. Rands, the following abstract of Professor Diller's report is considered pertinent to the discussion:

"The rocks of the canyon walls are of four forms, volcanic breccias, lava sheets, volcanic dikes and terrace gravels. * * * The volcanic breccia (bed-rock) is made up of unassorted angular fragments of lava, andesite and basalt of various colors, ranging in size from dust particles and grains of sand to large rock fragments many feet in diameter.

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* * * Sheets of solid non-fragmental lava forming part of the bed-rock and outcropping on the slopes of the canyon occur within and between the great sheets of volcanic breccia. Some of the lava sheets are basalt * * * generally very porous. * * * Nearly vertical dikes of basalt cut up through the sheets of volcanic breccia and lava, and their outcrops on the surface have the direction of N. 65° W. approximately parallel to the general course of the canyon. * * * There is a set of parallel joints, the open cracks of which cut up through the volcanic breccias and sheets of lava about vertically in a direction approximately parallel to the course of the canyon. * * * Such joints may be of considerable extent and form important openings for the circulation of water. Well developed joint cracks of this system were not seen in the exposed bed-rock of the dam site, but they may be expected, and should be carefully looked for where the bed-rock is covered with soil or gravel. It is especially significant that the dikes are approximately parallel to these joint cracks and suggest that these joint cracks may extend to great depths. * * * From the nature of the volcanic breccia, which forms by far the greater part of the canyon walls, it is evident that the drill cores will differ from one another very much when compared. Where the drill goes through a sheet of lava or a large solid fragment it will yield a good core, but where it penetrates the finer material (the volcanic ashes, in which the fragments of all sizes are imbedded), the core fails, the material is pulverized by the drill and washed away, and yet the extent of this material that is washed away is of the greatest importance, for it is the weakest element in the structure and the one which when saturated with water under pressure is most likely to become Engineers' 'soapstone'. Soapstone, properly so-called, does not occur in that region at all, but decomposed lava, volcanic ashes, and clay, all of which when saturated with water may become slippery and would be called 'soapstone' by Engineers, occur locally in the volcanic breccia. Large caverns and cavities, or pockets of loose earth and stones, are not to be expected in the volcanic breccia, but, owing to the manner of accumulation, there may be small openings, and the porosity of the rock is high. It is pervious to water, and for this reason similar material is used for making water coolers. The crushing strength of the volcanic breccia is, of course, small as compared with granite, limestone, and most other rocks, and this, taken in connection with its porosity and the possible existence of undiscovered joint cracks, seems to make a large reinforcement with concrete necessary, in order to furnish strength and prevent seepage as well as erosion.

"The conditions that confront the engineer along the Clackamas River in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade Range from the Columbia River in Oregon to the Feather River in California—one of the most important water-power belts in the United States—and the successful solution of the problem it presents at one point will greatly facilitate the work elsewhere."

At this time only one of the core-drill holes had been subjected to water pressure as part of the original programme of studying the porosity of the foundations. In a report to his clients, covering in-

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structions to the Field Engineer, Shirley C. Hulse, Assoc. M. Am. Soc. C. E., the writer said at this time:

"The result of this test shows that 25 gal. per min., at a pressure of 175 lb., have been pumped into this hole for several consecutive hours, and, while coloring matter was used, no points of escape were noted and the behavior of the water surface in the surrounding holes has been so small and erratic as to be negligible. It has been evident since the first holes were drilled that we are dealing with a very porous and permeable foundation condition which will require special treatment to render it suitable for the construction proposed, and the purpose of the pressure tests outlined was to indicate the extent of permeability and the general lines of leakage, so that we might best plan to make the foundation of the dam impermeable. * * *

"In the Middle West, where a number of masonry structures are built on sand or earth foundations, it is customary to cover the river bottom for some distance up stream with a layer of clay and depend on the silt in the river closing up all the fissures and paths of seepage, but it would be very unwise to depend on this method in such a construction as we have under consideration. * * * It is necessary to solidify and render impermeable the formation immediately beneath our proposed dam before we are justified in building a dam at this site.

"It is evident that impermeable foundations are desirable: (a) to minimize the possibility of upward pressure under the base of the dam superstructure (this being a solid concrete dam); (b) to prevent percolation under the dam which might lead to sufficient erosion to involve undermining the structure; (c) to overcome any structural weakness due to the original geological formation, and properly provide a support for the superimposed load; and (d) to avoid waste of water (the chief power asset of the Company) from the reservoir, through, under, or around the dam, instead of through the turbines where it creates K. W. H. available for sale.

"It is * * * evident that we cannot afford to build the plant and find out afterward whether we can make the foundations impermeable, but that we should adopt a programme whereby we may hope absolutely to demonstrate our ability to render the foundations impermeable before the superstructures are built. * * * The general idea involves drilling a double line of holes under the heel of the dam and forcing into each hole grout of such consistency as to percolate through the permeable substructure and solidify it absolutely throughout the entire length of the superstructure, thus making the foundations absolutely solid and also providing the equivalent of an absolute cut-off wall. * * * Cement grout should be pumped into these holes and after time has been allowed for hardening, a third line of holes should be drilled midway between the first two lines and tested under water pressure. If water pressure is applied at or slightly above the hydrostatic pressure to which this rock would be subjected by the water in the reservoir after the completion of the dam, and it is found impossible to force any amount of water through the holes, we may feel reasonably certain that the cement grout has proven effective in making the entire foundation impermeable. * * * The test * * * may show a very large quantity of leakage, and this will simply mean that more holes would

have to be drilled and pumped full of grout. * * * The programme to be followed will be necessarily indicated to a considerable extent by the experience obtained as the work progresses. * * * In order to avoid the uncertainties consequent upon an arbitrary determination of the spacing * * * of these grout holes, as well as of their depth, it appears expedient to select a convenient site where the cores indicate as difficult a formation to make tight as any on the job and to use this site as an experimental laboratory. * * * It seems probable, however, that this experiment may be made as part of the preparation of dam foundations, and the total cost of the work thus kept at the minimum. * * * We still regard it as essential that our ability to make the foundations tight be absolutely demonstrated before the work is started on the dam, and the experimental programme outlined provides for such demonstration." Mr. Rippey.

This report included detailed descriptions and sketches unnecessary to reproduce here.

At about this time the company acquired from other interests the "River Mill" property, below Estacada, and for business reasons decided to push a development there to early completion. The programme adopted by the owners of the properties provided that the detailed surveys, foundation tests, and preparation of detailed plans be made by the local organization and submitted to the writer, as Consulting Engineer, for his criticism and approval. The instructions issued specified: "This particularly applies to the foundation, as no chance or risk must be taken of locating a dam upon unsuitable foundations".

The general character of development adopted was similar to that already planned for the higher-head, upper project, except that the hollow reinforced concrete type of dam, which the writer had selected for two earlier successful developments in New England, was adopted on his recommendation, and the grouted preparation of foundations on the general plan in progress at the upper site was prescribed.

The programme adopted in the interest of saving time, with the writer's hearty approval, was soon violated in many respects by the local organization (the personnel of which has since changed). The construction programme being followed was found to be such as the writer could not approve, and he, therefore, advised his clients that the modifications locally made in the foundation treatment were "beyond the limits of the precautions which should be taken in such an undertaking, and while no one can claim with any assurance that failure will result, I cannot conscientiously approve the programme adopted, nor assume any responsibility in connection with it."

The writer also stated, in another communication:

"The successful treatment of the foundations is dependent to a large extent upon the detailed methods adopted, but it is also dependent upon the spirit with which this work is carried out. Your organization has already expressed the opinion that detailed studies and

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grouting are unnecessary, and, further, that detailed programmes prepared in advance of actual exposure of foundations are useless.

"We therefore feel that, aside from the engineering questions involved, your organization has not a proper appreciation of the subject; and further, any programme that may be insisted upon which is not in accord with their ideas would not be carried out in the proper spirit."

The foundation work had been held up for several months because of the writer's inability to approve what was proposed and the unwillingness of the local organization to follow the programme ordered, when the President of the Company suggested that John F. Stevens, M. Am. Soc. C. E., late Chief Engineer of the Panama Canal, be asked to pass upon the questions at issue. As a result of the opinions expressed by Mr. Stevens* (who was practically an arbitrator selected by the local parties), the owners of the property were confirmed in their support of the judgment of the writer, and insisted on the work being properly executed. So much time had been wasted, however, and the urgent need for power was so great, that it became necessary to carry along the grouting work in connection with actual construction of the superstructures in the best possible manner under the circumstances, and the conditions were not ideal for the development of the full possibilities of the use of grout.

The reversal of the local authorities and their reluctant acceptance of the ordered programme produced an attitude which resulted in unnecessary difficulties, obstructions, and personal controversies on the ground, finally leading to another change of Resident Engineers, for

*By the time the situation was formulated for presentation to Mr. Stevens, the local organization had developed a programme for the drilling, testing, and grouting of foundations, coincidently with or subsequent to the construction of the dam and other superstructures, instead of prior thereto, and his formal report, therefore, related largely to a comparison of the methods of execution, rather than to the accepted fundamental necessity for grouting the foundations. The following extracts from his report, however, indicate clearly his attitude on the larger question and his appreciation of the importance of foundation treatment:

"The limited number of test pits and borings which have been made shows clearly that, as far as concerns the area they covered, no well-defined, extremely hard masses of rock exist—that the rock is soft and shades away into clay, the latter appearing under superficial examination much like a half-burned brick. The whole formation, to the non-geologically scientific man, seems to be a mass of soft conglomerate, * * *

"The writer regards it as unfortunate that more test holes were not put down. * * * Enough testing, however, has been done to establish the fact that seams, or fissures, or cavities, do exist where certain of the holes were put down, and therefore it is only fair to assume that they exist where no testing was done; in fact, the only safe assumption is that they exist all over and through the formation, but to what sizes and shapes no one can say. There is no doubt in the mind of the writer that all such seams or cavities can be filled and sealed to exclude water by the proper introduction of cement grout. * * * If liquid cement under 200 lb. pressure cannot be forced into the material, the water under 35 lb. cannot find its way, and the writer concurs in such opinion. * * * Grout to be forced in by compressed air under 200 lb. pressure, the flow to be continuous from start to finish. This the writer believes to be the most practical and best method. * * * the writer believes that the material is, over the greater part of the area, porous enough so that under the heavy pressure the grout would form a practically continuous body, or curtain, from 12 to 15 ft. in thickness. * * * No living man can guarantee the absolute success of the grouting when it is placed."

purely personal reasons. Under all the circumstances, it reflects great credit on the ability and adaptability of all the engineers engaged on the work that they were able to attain as good results as those described by Mr. Rands in the face of aggressive lack of sympathy displayed by the local organization toward the foundation programme. Mr. Rippey.

As his clients loyally supported the writer's judgment in these matters, he regrets the necessity for discussing them in this connection, and at this late day. The Profession, however, is entitled to a knowledge of the actual conditions of the execution of the work, in order that other engineers may not be misguided by the results and by the opinions expressed. Moreover, as the writer's connection with the undertaking has been thus publicly recorded by Mr. Rands, the foregoing explanations properly become part of the record.

It must be understood that although the writer in 1910 declined to be considered responsible for results, in view of the conditions developed through the continued disobedience of governing instructions, he has no reason to question the safety of the final construction, which was nominally completed along the ordered lines, although under many handicaps.

While the Estacada work was in progress, the company acquired other partly developed power properties, and the Upper Clackamas development was therefore postponed before any conclusive results had been reached in the grouting experiments at that site.

About 2 years ago, in discussing the Estacada Dam in relation to the original experiment, the writer stated:

"It will be evident that the question of uplift pressure under a dam of this type is practically negligible, and, as the static head created is only about 83 ft., the other risks incident to a permeable foundation become proportionately smaller. As an indication of the porosity of the foundation material, it was found that the average leakage through 43 holes each 50 ft. deep (individually tested) was at the rate of 82½ gal. per min. under an average pressure of only 17.7 lb. per sq. in. The problem here becomes that of reducing the permeability to reasonable limits to overcome any structural weaknesses, prevent objectionable erosion and limit waste of water, without insisting upon the degree of tightness which would be necessary if the upward pressure under the dam base were a factor. The work was very urgent and the construction programme was modified in numerous respects to meet the exigencies which developed, but the dam was completed in November, 1911, and the foundation treatment appears to have met the practical requirements, although it was not pursued to the extent required to entirely eliminate upward pressure (had a solid dam been adopted).

"The holes were not uniformly spaced throughout the foundation, being located as experimental tests and the related construction work indicated best under the circumstances, and while the dam has been successfully completed and the plant is in regular operation, it is probable that the foundation treatment could be more thoroughly and

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economically effected in another case if taken up vigorously during the preliminary stages of a development so as to be independent of construction complications. Moreover, the practical experience obtained should greatly facilitate adapting this method to another development. It will be quite obvious, however, that each site will have its own peculiarities, and radically different treatment may be required."

Although the writer thus recognized the incompleteness of the grouting results attained as a result of the conditions described, and the degree of subordination of the grout work to the construction programme, which existed, it does not appear that the results were as negative as might be inferred from some of the personal conclusions reached by Mr. Rands. The very large quantity of cement introduced into the foundations necessarily represents an improvement in the formation to that extent, aside from any interpretation of pressure-test results, and, as Mr. Rands states, tests made after completion disclosed no leakage of water past the cut-off as the result of a hydrostatic head of from 80 to 90 ft. above the dam.

Although the final official report of Mr. Frank R. Fisher, who was Resident Engineer in charge during the period of completion of the development, indicates clearly that the grout did not produce impermeability over certain limited areas, it shows very definitely the practical sufficiency, for the purpose, of the results attained, as indicated by the following quotations therefrom:

"The excavation of the trench for the supplementary cut-off wall, in the locality where grouting had previously been done, afforded an excellent opportunity for observing its effects, as many seams were exposed, and all of them proved to be well caulked with cement. None of any magnitude were observed, the largest ones averaging about ½ in., and varying from that down to $\frac{1}{16}$ in., the smaller ones also having taken the grout freely.

"There was a total of 29 proving holes put down on this [the right channel] section, and, discarding the tests on two of them that represented excessive leakage that was only local, the average for the remaining 27 was 3.6 gal. per hole. In material of this kind this was not considered excessive, and it is reasonable to assume that for all practical purposes the cut-off is effective on the right channel section.

"*Conclusions.*—The final conclusions to be drawn * * * may be stated as follows:

"*1.*—It is not feasible to accomplish an absolutely impervious cut-off by this method, in foundation material of the character that exists in this region. The method apparently is effective in closing up seams and crevices of appreciable size, as all such exposed by the excavations made subsequent to the grouting were tightly caulked with cement.

"Where seepage occurs through a more or less porous formation, or through veins of compact broken rock mixed with sand or a certain kind of clay rock, very much seamed, but with the joint or cleavage faces in close contact, all of which have been observed to exist over very

limited areas in this locality, the grout has but slight effect in benefiting the condition. Material of this character acts as a filter, the cement remaining in the hole, and it cannot be diffused throughout the surrounding mass. Mr. Rippey.

"*2.*—While the grouting did not result in an impervious cut-off, the amount of possible seepage through the foundations was very much reduced thereby and it may be considered tight to the extent that erosion and appreciable waste of water would not occur.

"*3.*—The tests on the two holes put down inside of the dam, after the filling of the pond, indicate the presence of some upward pressure in the foundations. It is questionable, however, whether this could be prevented, even with an impervious cut-off 50 ft. in depth, for the indications are that the material in this particular locality is such that water would travel downward and pass under any depth of cut-off that it would be practicable to accomplish.

"*4.*—As it has been proven that the foundations cannot be grouted absolutely tight, it is obvious that but little good is accomplished in attempting to reduce still further what might be termed a reasonable rate of seepage by the introduction of additional holes."

The interest manifested in the Clackamas grouting experiments and the successful adoption of similar methods elsewhere clearly demonstrate the fundamental merit of this method of treating deficient foundations. Indeed, in some cases it appears to the writer to afford the only practicable means of rendering certain formations satisfactory for the construction of high solid masonry or concrete dams. The problem of eliminating upward pressure under the base of solid dams, the significance of which was emphasized in the discussion of a paper* by the late C. L. Harrison, M. Am. Soc. C. E., is peculiarly susceptible of solution by the grouting method in certain formations. In the course of the discussion on that paper, Arthur P. Davis, M. Am. Soc. C. E., said:

"Recent experience has shown the feasibility and efficacy, in some cases, of closing the crevices in the foundations wholly or partly by grouting them under pressure. This was accomplished successfully at moderate cost on the Ashokan Dam, and on several others of recent construction. The most striking instance of this kind which has come to the writer's attention is the Clackamas Dam, in Oregon, which was built on a foundation of semi-indurated volcanic ash, which was checkered in all directions by innumerable fissures, and, furthermore, was so soft that percolation was likely to cause destructive erosion. A triple line of holes was grouted along the up-stream toe of this dam, and recent information is that, since the dam has been in use, no perceptible percolation has taken place.

"The effect of such grouting is not easy to foretell, and, like all other underground conditions, must be estimated with extreme caution."

Incidentally, it was the failure of the local organization to recognize the necessity for this extreme caution, as emphasized by so ex-

* "Provisions for Uplift and Ice Pressure in Designing Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 142.

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perienced an authority as the Chief Engineer of the U. S. Reclamation Service and insisted on by the writer, in connection with the Clackamas work, which led to the unfortunate lack of sympathetic co-operation at Estacada.

A private letter from certain foreign engineers who had inquired about the Clackamas grouting experiments, from which the writer does not feel at liberty to quote directly without permission, reported the complete success of grouting to create an impermeable mass of a defective formation through which water freely escaped from a reservoir. It appears that the greatest success was secured by using a very dilute grout mixture, starting with quite low pressure, and finishing with pumped injection, whereby a certain amount of shock was applied which tended to overcome any temporary obstruction. It was demonstrated that the grout, which set well, traveled more than 100 ft., in plan, and the results appeared satisfactory in the highest degree.

In a recent article in a technical journal,* D. W. Cole, M. Am. Soc. C. E., Project Engineer, U. S. Reclamation Service, describes the application of the grouting method to the foundations of the Lahontan Dam of the Truckee-Carson Project. The writer understands that Mr. Rands was in direct charge of the grouting work (following the completion of the Estacada work), and Mr. Cole describes the results as "excellent", evidently considering, in advance of the completion and filling of the reservoir, that they were successful in accomplishing their purpose. He refers to the fact that it was proven that the grout hardened in place and gives evidence of the effectual sealing of fissures and seams and the solidification of the foundations adjoining the grouted holes.

Another article, in the same journal,† describes the grouting work done on the foundations of the Olive Bridge Dam of the Ashokan Reservoir, part of the Catskill water supply project for New York City. Two definite planes of slight seepage, 40 and 60 ft., respectively, below the creek bed, were found. It appears that 1 439 cu. ft. of thick grout were injected (under comparatively low pressure), of which it was estimated that 1 100 cu. ft. entered the seams. No proving tests seem to have been made.

Ample evidence exists to demonstrate the broad possibilities of the grouting method in its application to the solidification of defective foundations, and it remains for the engineer in each case to adapt it to local requirements. Of course, no general engineering method or formula can eliminate the need for experienced judgment in its detailed application, and, in work of this character, sympathetic co-operation is necessary, down to the humblest participant in its execution.

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* *Engineering Record*, March 29th, 1913.

† *Engineering Record*, April 8th, 1911.

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The writer believes that the grouting method affords many economical possibilities in water conservation and power development. It may permit the creation of an operating head entirely by a high dam, instead of by a low dam and a long, and perhaps leaky, flume; it may permit the selection of a site for a dam where the topography is favorable but where the natural foundation is impossible or not as satisfactory as at some poorer power location on the stream; and in many ways may be developed to render reservoir and power problems more flexible.

Even where the security of the construction may not be in question, the prevention of waste by percolation may amply justify the grout treatment to secure impermeability. At the Upper Clackamas, for instance, it was estimated that a saving of only 26 sec-ft. would justify an expenditure of $80 000, aside from any of the collateral advantages (there real necessities) of preventing the flow of water under the dam.

It is difficult to imagine any alternative method of foundation treatment in which the range of possibilities is as great as in grouting.

It will be observed that an essential feature of the grouting programme proposed by the writer is the ability to demonstrate by test, before building the superstructure, that the solidification of the porous material to a satisfactory degree of impermeability has been effected. The holes drilled by rotary drills for securing core specimens, and thus studying the foundations, were utilized later for the hydrostatic pressure testing, for washing out loose and soluble material, and for the introduction of grout under pressure. In the peculiar and exceedingly variable material encountered, it was found that in some cases communication existed between holes as far apart as 70 ft., and in other cases grout could not be made to permeate the material sufficiently unless the holes were very close together. The result was that a greater number of grout holes were required than was tentatively assumed without any experimental knowledge whatever, and a greater number than were required for core studies alone. This fact suggests that, in other applications of this method to large undertakings, only sufficient core holes be drilled by rotary drills to permit proper studies of the geological formation and general foundation structure, and that the remaining holes be sunk purely for grouting and testing purposes by apparatus which will perform the work more rapidly and economically.

The nature of the operation involved in securing cores is such as to necessitate slow progress and relatively high cost per foot of drilling, and when the writer reached a point on the original work where he was not specially interested in securing additional core samples, and where the results showed the need for closer spacing of grout

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holes than was thought from the general appearance of the rock and its permeability to clear water would be necessary, he considered the use of drills which would perform the remainder of the work more expeditiously and at less cost than was possible with rotary drills made primarily for securing cores.

The average rate of progress with the core drills was about 13 ft. per 10-hour shift, and it was evident that if the further work was confined to plain drilling it could be done much more rapidly. It appeared that the future drilling might best be done with percussion drills with guided rods arranged so as to drill holes 6 in. in diameter and about 50 ft. deep, approximately plumb, at a high rate of speed. A number of drills were found on the market which could be mounted to accomplish this work, but there was great difference of opinion among the manufacturers as to the possible rate of drilling in this material, estimates running from 25 to 150 ft. per 10-hour shift. At this juncture the development was postponed, for the business reasons stated, and the subject was not pursued further, but there is evidently opportunity for great saving along these lines.

The costs of drilling and grouting at the Estacada Dam, as given by Mr. Fisher in his final report, and by Mr. Rands, on a somewhat different basis, in his paper, and the cost of grouting given by Mr. Cole for the Lahontan Dam, all reduced to a unit basis, are given in Table 5.

TABLE 5.—COST OF DRILLING AND GROUTING AT ESTACADA AND LAHONTAN DAMS, PER LINEAR FOOT OF COMPLICATED WORK.

Labor and materials.	Estacada Dam.		Lahontan Dam.
	Fisher.	Rands.	Cole.
Labor, drilling	$0.58	$0.59	$0.93
Labor, grouting	0.18	0.18	0.29
Cement	0.12	0.12	0.31
Repairs and supplies	0.17	0.17	0.23
Plant	0.30	.....	.....
Plant depreciation	.....	0.15	0.35
Power	0.05	.....	0.03
Other items	.....	.....	0.94
	$1.40	$1.21	.....
Salvage on plant, Credit	0.17	.....	.....
Direct cost	$1.23	$1.21	.....
Total field cost	.....	.....	$3.08
General plant, etc.	0.32	0.45	0.12
Coffers and pumping	.....	0.15	.....
Engineering and superintendence	.....	0.19	0.27
Clerical and office	.....	.....	0.10
Total cost per foot	$1.55	$2.00	$3.57

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It is of interest to note that the cost of cement per foot of hole at Lahontan was more than 2½ times that at Estacada; examination of the data, however, indicates that cement cost about $2.77 per bbl. at Lahontan and only $2.20 at Estacada. The actual quantity of cement used per foot of drilled hole appears to have been as follows:

Estacada.—34 038 ft. of holes: 1 942 bbl. = 0.057 bbl. per ft.
Lahontan.—2 593 ft. of holes; 1 174 sacks, say, 294 bbl. = 0.113 bbl. per ft.

In his final Estacada report, Mr. Fisher says: "The holes put down with the diamond drills cost about one-third more per foot than with the shot drills."

The average rate of drilling at Estacada was 1.3 ft. per hour and at Lahontan about 0.75 ft. per hour.

The use of percussion drills for grout holes after all necessary cores are taken, as has been suggested, should materially reduce the total drilling cost on a large job, and should also reduce the time element, which is of prime importance in most river work.

The writer has information concerning other successful grouting in dam foundations in a foreign development, but, unfortunately, is not at liberty to communicate it at this time. Although the method described has inherent possibilities possessed by no other system, cases may arise wherein the impregnation of an extended rock mass with grout is not necessary, but where a definite and absolute sub-surface cut-off is required. There are many difficulties and limitations incident to the construction of a cut-off wall in an excavated trench for this purpose, especially for work under a high head where the desired depth is great, and, in connection with the consideration of the grouting scheme, certain alternatives for such limited applications have been developed. No opportunity has occurred for testing these, but some suggestions concerning them appear to be pertinent to this discussion, and may lead to the further development of foundation treatment methods.

Fig. 16 illustrates the general features of a suggested cut-off; this plan contemplates drilling a single line of 6-in. holes on 9-in. centers, broaching the intervening webs so as to form a continuous slot, introducing interlocking steel piling to ensure a water-tight barrier to the passage of water, and firmly securing this steel curtain in place and backing it by concrete, in the manner shown. The introduction of this curtain would afford the assurance of a positive stop to the percolation of water, but would not provide for the solidification of the mass of the foundation so as to increase its bearing value. For this reason this method might be specially applicable to a stratified foundation where approximately horizontal seams or bedding planes containing clay or soft material constitute a source of danger only when subjected to erosion.

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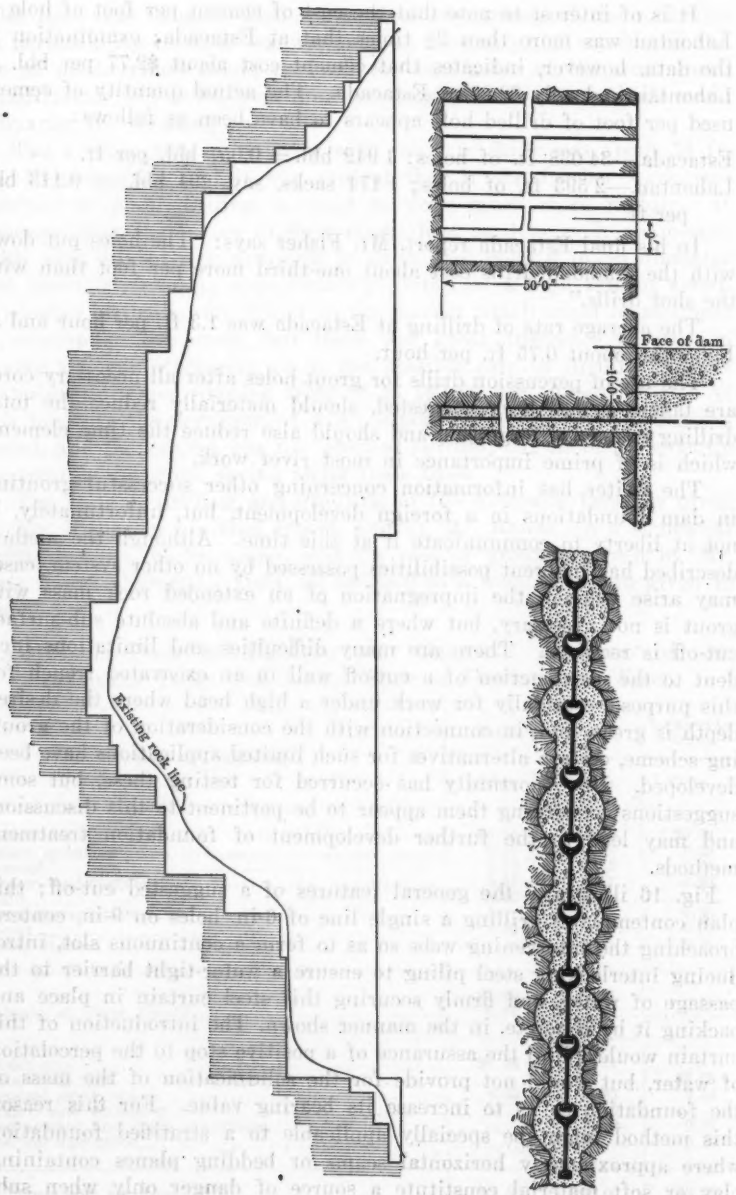

Fig. 16.

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In considering the proposed method of constructing an effective cut-off barrier of this type, the details of execution remain to be studied. There appears to be no doubt that any one of several makes of percussion drills is capable of drilling 6-in. holes 50 ft. deep at relatively high rates of speed, but in ordinary work the question as to whether such holes are straight and truly plumb has been given little consideration. In the proposed scheme, the necessity for inter-connecting the drilled holes by broaching or blasting out the intervening webs to form a continuous slot, necessitates a certain degree of parallelism of the adjacent holes, and, as such drilling is without precedent for holes of this depth, the drill manufacturers have naturally been unable to make any guaranties as to performance in this respect, but there appears to be no doubt as to the possibility of working out this feature of the problem. Incidentally, the manufacturers have nothing to offer in the way of a machine for directly channeling such a slot as required, and the slot seems to be physically, as well as in the conception of method, the natural development of a series of holes drilled reasonably close together.

As alternatives to the plan proposed, wherein the steel curtain provides the absolute cut-off within the limits of its depth, two modified schemes have been considered:

(a) Drilling a row of 6-in. holes, A, B, C, etc., on 9-in. centers, as shown by Fig. 17, Sketch 1, filling these holes solid with cement mortar or grout, and, after setting, drilling intermediate holes, 1, 2, 3, etc., which in turn would be grouted, thus forming a continuous concrete wall of 6-in. maximum thickness, if the holes are all plumb and parallel. There would be no way of determining, however, to what extent the concrete fills the spaces, and the degree of its impermeability when set, so that there would be no proof of having accomplished the desired results.

(b) Drilling a single line of 6-in. holes, A, B, C, etc., as before and forcing grout into them, which would be expected to permeate fully the intervening rock, as indicated on Fig. 17, Sketch 2.

The assurance of impermeability with this procedure would be even less than in plan (a) and both ideas were discarded as insufficient.

Inasmuch as the creation of a continuous slot 50 ft. deep is a novel proposition, the execution of the programme would require the most careful supervision, in order that detail methods might be devised which would insure the complete realization of expectations at minimum cost. In the first place, the drill should be mounted on a frame constructed so as to be easily and quickly moved from hole to hole and maintained plumb under the powerful high-speed stroke contemplated, to the end that the holes should be as true and plumb as is practicable. When the holes in a given section are completed, the

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broaching of the intervening webs of rock may be effected by replacing the 6-in. drilling bit with a broaching bit and disconnecting the rotating devices used for driving round holes, so that the bit may be partly rotated by hand, as required by the varying conditions

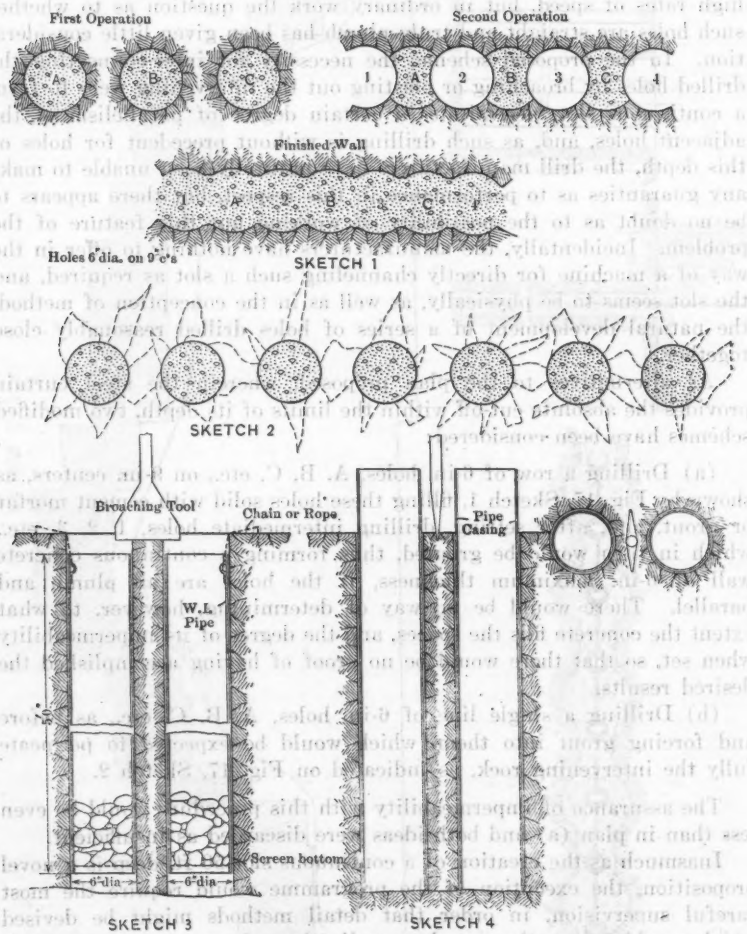

FIG. 17.

encountered. It seems possible that in some instances the original holes might deviate from the theoretical lines sufficiently to warrant drilling additional intermediate holes in places to facilitate the broaching, and it also appears possible that a limited amount of blasting might be justified in order to break out irregular webs into the con-

tinuous slot; these, however, are details to be worked out in the field during the progress of the work. Mr. Rippey.

In drilling the original holes there should be no difficulty in washing the cuttings to the surface in the usual manner, so that each hole on completion should be practically clean. There is opportunity for some ingenuity in experimenting with the broaching process, both as to the cutting and the removal of the cuttings, and it is obviously important to be assured that these cuttings are removed so as to interpose no obstacle to the introduction of piling or the complete filling of the slot by solid concrete. This cleaning out of the broach-

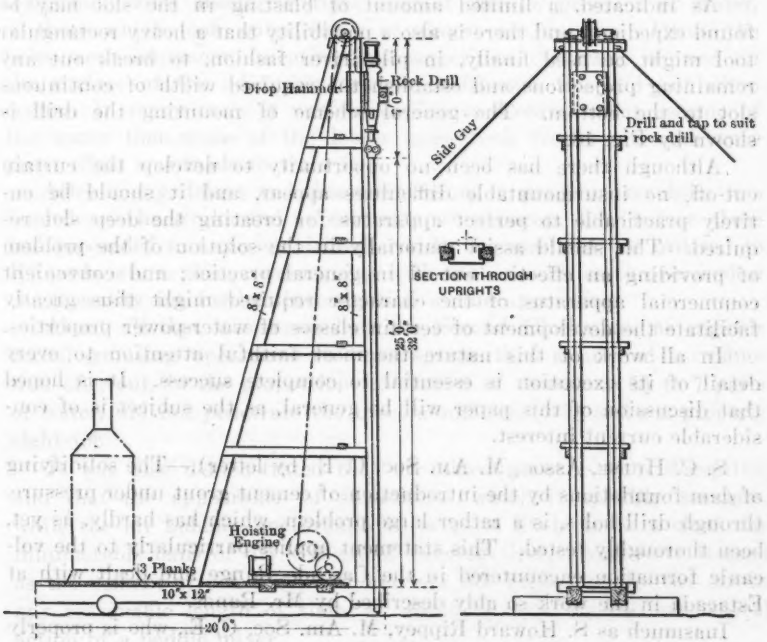

FIG. 18.

ings may require some experimentation to secure the best results, but one method which suggests itself is illustrated in Fig. 17, Sketch 3. Two standard wrought-iron pipes, having an outside diameter of $5\frac{9}{16}$ in. and perhaps 5 ft. long, are indicated, with the lower ends covered by a close-meshed wire netting which will pass water, but not cuttings. These tubular buckets, supported by chains with their tops just below the lower limit of stroke of the broaching bit, would collect almost all the cuttings, being raised and emptied when full. This would leave about 5 ft. at the bottom of the holes which could not be treated in this manner, but a shorter pair of buckets might be used at the

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bottom, and the final cuttings might be washed out as completely as possible.

Another method considered, although probably more expensive and less favorable for the rapid action of the broaching bit, is illustrated in Fig. 17, Sketch 4. This involves placing temporary pipe casings in two adjacent drilled holes to guide the broaching tool and prevent its cuttings from dropping into the clean, drilled holes. In this plan the broachings from the web would be washed to the surface by a water jet in the same manner as in the drilled holes; a few trials should determine the most effective method.

As indicated, a limited amount of blasting in the slot may be found expedient, and there is also a possibility that a heavy rectangular tool might be used finally, in pile-driver fashion, to break out any remaining projections and establish the required width of continuous slot to the bottom. The general scheme of mounting the drill is shown by Fig. 18.

Although there has been no opportunity to develop the curtain cut-off, no insurmountable difficulties appear, and it should be entirely practicable to perfect apparatus for creating the deep slot required. This should assist materially in the solution of the problem of providing an effective cut-off in general practice; and convenient commercial apparatus of the character required might thus greatly facilitate the development of certain classes of water-power properties.

In all work of this nature the most faithful attention to every detail of its execution is essential to complete success. It is hoped that discussion of this paper will be general, as the subject is of considerable current interest.

Mr. Hulse.

S. C. HULSE, ASSOC. M. AM. SOC. C. E (by letter).—The solidifying of dam foundations by the introduction of cement grout under pressure, through drill holes, is a rather large problem, which has hardly, as yet, been thoroughly tested. This statement applies particularly to the volcanic formation encountered in the Cascade Range and dealt with at Estacada in the work so ably described by Mr. Rands.

Inasmuch as S. Howard Rippey, M. Am. Soc. C. E., who is properly accredited by Mr. Rands as the originator of the plan to grout a cut-off curtain across the (Second Clackamas) site several miles above Estacada, will doubtless take part in the discussion of this paper, the writer will confine himself rather to certain practical aspects of the grouting problem.

When the writer went to Oregon for Mr. Rippey in the fall of 1909, he had been given clearly to understand that his work at the Second Clackamas was to be very largely of an experimental nature. Mr. Rippey dwelt at length on the fact that his ideas (as generally expressed in the extract from the paper by Mr. Schreiber on page 10*

* *Proceedings*, Am. Soc. C. E., for January, 1914.

of the paper) dealt with something yet to be proven, and he, in spite of his strong belief in the ultimate result, insisted that his own attitude toward the results of the experiments to be made, would be most conservative. Mr. Hulse.

The experiments at the Second Clackamas were halted (by the postponement of the work for business reasons) before any positive results were obtained, as regards the actual efficiency of grouting. Somewhat more than 5 000 lin. ft. of core-drill holes were put down, and these ranged from 40 to 250 ft. in depth. Many and exhaustive pressure tests were made in these holes, and they ranged, from a few which were absolutely tight against 200 lb. pressure of water, to those which took water freely at almost no pressure. Often the holes were intercommunicating and sometimes the water showed on the surface of the adjoining territory—although this did not often occur. In many cases it was impossible to discover what became of the water that some of the looser holes took freely, although very great effort was made to trace it.

After many delays and disappointments, a grouting machine was landed on the work, and experiments were begun, but, before these had proceeded to any very definite result, the machine was ordered to Estacada where things were going with such a rush as to overshadow the necessity for further investigation at the upper site.

About all that was learned at the Second Clackamas, as regards actual grouting operations in breccia, was that a hurried introduction of grout, in the proportions of from 1 of cement to 1½ to 7 parts of water, did not penetrate the breccia sufficiently to make a properly tight job.

It was also demonstrated that air-mixed grout is very deceptive, and that the appearance of smoothness which it speedily assumes, while the air bubbles up through it, is quite likely to be a cloak for innumerable balls of dry cement of varying sizes, which have formed as the cement struck the water and are not broken up by the air in any reasonable length of time, as would be done by the mechanical action of a paddle mixer.

The openings in breccia range from the tiniest of pores into which cement may only be forced, if at all, with the greatest care, but through which water under high pressure will find its way, to cracks and cavities of considerable size. Any one who has had experience in cleaning the valves of a grout machine which has become plugged with a ball of dry cement under pressure may readily realize the facility with which badly mixed grout may close entrances to openings and thus prevent them from being properly filled.

The writer's experience with surface leaks at the upper site had not been encouraging, up to the time the experiments were discon-

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tinued, but, on subsequent work, he has learned how to handle these, as will be detailed later.

To proceed to the conclusions arrived at on page 36* of the paper:

(1) The advisability of a concrete cut-off wall under the foundation of a dam is open to serious question, and the writer has heard more than one able and experienced engineer declare against the practice. To put in such a cut-off, a trench must first be opened. If this is done by hand, in any material capable of bearing a great dam, the work is likely to be slow and costly. If it is done by blasting, and particularly in breccia, where the use of a channeler is said to be impracticable because of the nature of the material, there is great danger of doing almost irreparable damage to the surrounding material, and this is exactly what happened in the left channel at Estacada, where the breccia adjoining the heel-trench was shaken and cracked in all directions by the blasting. The writer believes that grouting, properly done, may be used as a satisfactory substitute for a concrete cut-off, and to the top of the foundation rock.

(2) The testing of drill holes is such a fascinating pursuit (and the writer speaks from experience) that it may easily be carried beyond the limits of practical usefulness. However, the tank method is quite preferable where much testing is to be done.

(3) This method of procedure might readily entail an almost prohibitive amount of shifting about, and, if carried to its logical conclusion, might even lead to a complete and wholly unwarranted interruption of the grout and drill work. More will be said about this in connection with Conclusion (5).

(4) This conclusion is very true, and emphasizes the desirability of a grouting machine which may be operated by water pressure instead of by air. The following of water into the hole, behind the grout, would not be objectionable, whereas the air is a nuisance, and has to be closely watched, in this respect. The writer has hopes of developing such a grouting machine.

(5) and (3) There are two sorts of grouting which may be accomplished: plugging, and sedimentation or silting. There are several ways of going after subterranean territory, through the medium of a drill hole, in the hope of making that territory impervious to the passage of water under pressure, and the writer wishes now to present the method which seems to him best suited to the penetration of every opening of sufficient size to admit cement particles in the form of grout. This method is practically irrespective of the nature of the rock to be grouted.

The first necessity is a reliable driller, who realizes his responsibilities and will enter into the spirit of things. Fortunately, such drillers are not the exception, although one does occasionally meet with

* *Proceedings*, Am. Soc. C. E., for January, 1914.

an experience such as has been mentioned by Mr. Rands in connection with his search for cement in the cores. Mr. Hulse.

Casings should be set beyond the possibility of their coming loose. If this is impossible—as is sometimes the case in soft breccia—a mat of concrete, preferably to be incorporated later in the main structure, may be laid on top of and well bonded to the rock surface, at or near the heel line, and the casings set in this mat. The mat should be as narrow as possible, so that it will not close any more surface leaks than necessary.

If a core drill is used, the loss of drill water or the striking of a seam or crevice is the best indication of the time to test the hole. If a percussion drill is used, it may be well to test at stated depths—say, every 10 ft.—because the evidence of what is encountered by a percussion drill is not nearly so reliable as in the case of a core drill.

The point is that leaks should be grouted as soon as encountered. This is for the purpose of closing openings which would let the pressure out of the lower portions of the hole (yet to be drilled) were they not closed. In the absence of special equipment for testing purposes, charges of air or water blown through the grouting machine will answer perfectly well to indicate how freely the hole may be expected to take grout, and to show up any leaks on the surface.

The first grout introduced should be quite thin—say, 1 part of cement to 30 parts of water—and it should be absolutely free from lumps of any sort. Pressure should be applied gradually, and the instant a surface leak shows, the pressure should be dropped to such a point that only water (perhaps colored with cement) flows from the leak. This is the starting point for a successful closure of that leak by a process of sedimentation, as contrasted with the possibility of plugging it immediately by using thick or lumpy grout. From this time forward it is a matter of patience and of judgment in the application of pressure and the thickening of the mixture, and a great deal of patience is worth a very little thickening, as judged by the final results. Cement is never wasted by this process of "slow grouting" as long as it does not appear at the surface, and a careful nursing of the pressure will deposit the cement underground where it is wanted. In time—sometimes very quickly—the leak fills, and, as the resistance increases, more pressure is applied, until finally the hole refuses at the maximum pressure available. An exploration of many surface leaks thus "slow grouted" under the writer's direction, has disclosed the fact that they had been tightly filled with cement, and so very tightly as to preclude any subsequent opening due to shrinkage, as would have been the case had they been merely filled with the thickest grout that could have been run through them.

There is nothing in this sort of treatment to preclude the almost immediate washing out of the drill hole (after refusal) and the con-

Mr. Hulse.

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If a core drill is used, the loss of drill water or the striking of a seam or crevice is the best indication of the time to test the hole. If a percussion drill is used, it may be well to test at stated depths—say, every 10 ft.—because the evidence of what is encountered by a percussion drill is not nearly so reliable as in the case of a core drill.

The point is that leaks should be grouted as soon as encountered. This is for the purpose of closing openings which would let the pressure out of the lower portions of the hole (yet to be drilled) were they not closed. In the absence of special equipment for testing purposes, charges of air or water blown through the grouting machine will answer perfectly well to indicate how freely the hole may be expected to take grout, and to show up any leaks on the surface.

The first grout introduced should be quite thin—say, 1 part of cement to 30 parts of water—and it should be absolutely free from lumps of any sort. Pressure should be applied gradually, and the instant a surface leak shows, the pressure should be dropped to such a point that only water (perhaps colored with cement) flows from the leak. This is the starting point for a successful closure of that leak by a process of sedimentation, as contrasted with the possibility of plugging it immediately by using thick or lumpy grout. From this time forward it is a matter of patience and of judgment in the application of pressure and the thickening of the mixture, and a great deal of patience is worth a very little thickening, as judged by the final results. Cement is never wasted by this process of "slow grouting" as long as it does not appear at the surface, and a careful nursing of the pressure will deposit the cement underground where it is wanted. In time—sometimes very quickly—the leak fills, and, as the resistance increases, more pressure is applied, until finally the hole refuses at the maximum pressure available. An exploration of many surface leaks thus "slow grouted" under the writer's direction, has disclosed the fact that they had been tightly filled with cement, and so very tightly as to preclude any subsequent opening due to shrinkage, as would have been the case had they been merely filled with the thickest grout that could have been run through them.

There is nothing in this sort of treatment to preclude the almost immediate washing out of the drill hole (after refusal) and the con-

Mr. Hulse.

tinuance of the drilling to the next point where grouting may seem advisable. Holes grouted with thick grout will frequently take more grout, if re-tested within a few hours after the first refusal at maximum pressure, but it is very exceptional when one succeeds in making any impression whatsoever in re-testing a hole which has been "slow grouted" as just set forth. In the first case, the openings are plugged with a mixture which subsequently shrinks; in the second case, they are silted full so tightly as to preclude shrinkage.

As an illustration of the results to be gained by patience, the writer once grouted for eight continuous days and nights on a hole that took somewhat more than 200 tons of cement. For several days, thin grout was literally poured into that hole—much of it by gravity—and the grout was kept thin until the hole showed signs of closing up. Had Conclusion (5), as stated by the author, been followed, the operation would doubtless have ended much sooner, but the result (the hole, thus far, has proved to be tight against a head of 130 ft.) might have been quite different. Incidentally, of the more than 200 tons of cement used, perhaps a dozen sacks were wasted through the leaks which developed—surely not a high price to pay for placing where it was wanted the rest of the quantity used.

There is good evidence that Portland cement sets very slowly—if at all—after it has been introduced underground in this fashion, but, should the process of grouting necessarily entail an assumption of the necessity for the setting of the cement used? Rather should it not be looked on as an enforced closing of openings or passages by the silting or sedimentation of an insoluble material, introduced under an ultimate pressure greater than any which might subsequently tend to dislodge it? If the cement does set up, so much the better.

If the suggestion of the author, in connection with the grouting done at Estacada, that: "* * * so far as this job is concerned, that over a portion of the cut-off no grouting was needed, and that over those parts where it was needed it did little good", is to be sustained, what of the 1 920 bbl. of cement that were used? It is hardly to be assumed that all this cement went to close the cracks opened by the blasting of the heel-trench and to waste in dealing with surface leaks. That portion of the 1 920 bbl. which was not used for these two purposes must have gone somewhere in the foundation and have closed openings which would much better be closed than left open—else, why grout, or why bother about cut-offs? The action of the two test holes which were drilled inside the dam and flowed water, as set forth by the author, indicate obviously enough that the grout as applied did not close the breccia tightly, or, that the water passed under the curtain and then rose; and the writer is strongly inclined to the former view of the matter, but does not feel that the evidence in the case is to be viewed as damaging to the grouting system of tighten-

ing foundations. Rather, he believes that the Estacada work shows remarkable results in favor of the system. We have here, in the beginning, a case of a conservatively worked out scheme which is to be based on experiment and subject to experimental proof. Before anything much has been learned from the experiments, they are interrupted and a practical application is made, and under circumstances by no means favorable to a fair trial. Aside from the blast-shattered area which complicated the situation, the grouting work at Estacada was continually interfered with and hustled about, and hurried beyond the possibility of proper conduct, by the exigencies of rush construction; and, in the opinion of the writer, the engineers in charge of the grouting did remarkably good work under the circumstances. If, as the author states, no leaks of consequence have been observed below the Estacada Dam, may not a certain amount of credit be given to that part of the 1 920 bbl. of cement which was not wasted in handling surface leaks, and is it not conceivable that, had the work been less rushed, and had greater care with the grouting been possible, the flow in the test holes inside the dam might have been less? Mr. Hulse.

Grout work is likely to be at a disadvantage when brought into conflict with the man whose sole idea of the conduct of work is to set a new record for depositing a given quantity of concrete in the shortest possible time. For the grouting, there is little in evidence but a few small pipes—which may or may not connect with an underground opening which might, later, imperil the integrity of the concrete man's work, when the reservoir fills—and these small, insigficant pipes may even get in the road of the concrete man's buckets, and otherwise become very much of a nuisance.

If, therefore, it is possible to do the grouting and get the pipes out of the way before the beginning of actual construction, so much the better. If the grouting and construction must be carried on simultaneously, every effort should be made to keep them from interfering with each other; but, in case they must conflict, it seems rather superfluous to point out that, inasmuch as the integrity of the whole job will depend to a very great extent on the success of the grout work, the latter should be given at least "an even break" in the matter of precedence. An open cut-off trench, by its very presence, compels respect and consideration, but a line of 2-in. grout pipes, which may be expected to accomplish cheaper and better results than the cut-off trench, must frequently be aggressively guarded by the man in charge of them.

In any case, it is desirable that provision be made for drilling and grouting after the completion of the work, should later developments show the need for this later work. Usually, this may be done more at the cost of foresight than at the cost of trouble and expense.

The author's suggestion about the drilling of a row of holes close

Mr. Hulse. together and then broaching them into an open slot which may then be grouted, is open to the objections against blasting in the foundation material under the heel of a dam to be. If, however, this might be accomplished without damage, the writer would suggest that, before the holes are broached, they should first be carefully grouted so as to silt up the adjoining territory, and thus reinforce the thin solid cut-off to be made by filling the slot. Further, it would be most desirable, in filling the slot, to do so by introducing the grout at or near the bottom of the slot, and forcing it to rise slowly toward the surface. A succession of such operations, to be accomplished in a deep slot, by a gradual raising of the grouting pipes, would probably insure the tightest filling of the slot with the least chance of subsequent shrinkage and cracking of the material deposited therein. Without some reinforcement, which might be very difficult to place in a thin wall like this, the writer would not consider the scheme except in connection with a careful silting up of adjoining territory by first grouting the holes, as has been stated.

As a general proposition, the writer believes that the silting up of a foundation by grouting and by the action of the water impounded behind the dam, offers possibilities distinctly preferable to those of the conventional cut-off wall. In the case of a clear stream, like the Clackamas, it is usually practicable to make the water muddy by dumping or sluicing into it material from the banks, while the reservoir is filling, and thus close many or all of the openings missed by, or too fine for, the grout. The practice of shooting into foundation material under the heel of a dam is more than likely to make a lot of trouble—witness the left channel at Estacada—and the penetrating qualities of cement grout, properly handled, are very great. If the rock is good, why replace it with a concrete cut-off? If the rock is bad, and yet so solid as to necessitate blasting for its removal, how much better to silt it tight and leave it undisturbed?

Mr. Fisher. FRANK R. FISHER, ESQ. (by letter).—In the writer's final report as Resident Engineer in charge of the construction of the Estacada Dam, he stated, in discussing the grouting of foundations, that an impervious cut-off was not accomplished, that is, to the absolute degree had in view when the method was adopted. The varying and erratic nature of the formation made the solution of the problem largely experimental and one that could only be worked out by a thorough trial and without any positive assurance as to what the final outcome would be.

The writer does not consider that the method should be condemned or judged by the failure to obtain absolute tightness in the very peculiar material with which the engineers had to deal, but rather that credit should be given them for reducing the possibility of seepage to the extent that practically no uncertainty regarding the

safety of the foundations in that respect would remain. That this was accomplished, and the foundation material benefited by the treatment, he believes an analysis of the results, as stated in his report, will show. Mr. Fisher.

Mr. Rands arrives at the following conclusion: "that over a portion of the cut-off no grouting was needed, and that over other parts where it was needed it did little good." The first part of his statement is correct, in that over a considerable portion of the right channel and island sections, the average formation was found to be in good condition, not absolutely tight, but developing only a low rate of seepage under test. Naturally, this evidence was not apparent until it was disclosed eventually by the progress of the work, and then, in view of the ever-present uncertainty, due to the fact that occasional points would develop unexpected weakness, it was deemed advisable to follow out the original plan to completion. In the light of final knowledge acquired from the complete data obtained, it was evident that the grouting could have been omitted over certain stretches, and the number of holes on these sections materially reduced.

The remaining section, that across the left channel, is evidently what Mr. Rands refers to as: "other parts where it was needed it did little good". There was no question as to the need of some kind of treatment in this particular locality, as the material, from the standpoint of permeability, was the worst met with on the site of the dam. The water-pressure tests gave high rates of seepage, communication existed between holes located over a considerable distance, and also with the surface as many as 70 ft. from the point of testing. The holes took grout freely and, in the aggregate, several hundred barrels of cement were forced into the seams and crevices existing throughout the rock. As the individual seams were apparently of little width, the large quantity of grout taken would seem to indicate that it was diffused over a wide area.

Although the tests on the final proving holes, at the full depth of 50 ft., developed considerable seepage, it was very much less than the original. The most encouraging feature, however, was the very satisfactory results obtained for the first 30 ft. Over a length of cut-off of 70 ft., the aggregate of the seepage through thirteen holes, spaced approximately 6 ft. apart, when tested under a hydrostatic head slightly in excess of that created by the dam, was only 0.2 sec-ft., and it should be noted that this seepage was obtained by boring into the vitals of the foundations, and opening up and subjecting to direct pressure all points of weakness intercepted by the thirteen holes. It should also be kept in mind that this test was made on what was originally the weakest section of the entire cut-off.

It is also obvious that the test conditions were much more severe than could occur normally from the pond pressure, and, therefore, it

Mr. Fisher. is reasonable to conclude that the grouted cut-off is effective for a depth of 30 ft., and as this is reckoned from the bottom of an overlying 10-ft. concrete wall, the total effective cut-off is 40 ft.

As to the choice between a concrete and grouted cut-off for this depth, it is only necessary to point out that the high cost, difficulties of construction, and the element of time required—of vital importance in this case—would practically prohibit the use of concrete.

Evidence of the satisfactory final condition of the foundations was furnished subsequent to the completion of the dam and the filling of the pond by a thorough examination of the surface of the rock inside the dam, and the unwatered bed of the river immediately down stream. No springs or seepage could be observed, with the exception of a few very slight indications along the side of the left bank, and it is more than likely that these were from surface drainage.

The foregoing facts may not offer indisputable proof that the final satisfactory condition of the foundations was entirely due to the merits of the grouting treatment, but the evidence is in its favor, and it is entitled to all the credit that the results seem to warrant.

Mr. Coburn. H. L. Coburn, M. Am. Soc. C. E. (by letter).—Mr. Rands is to be congratulated on the thorough manner in which he has presented the problem and attempted solution in the matter of the cut-off under the Estacada Dam. The information contained in this paper will be of great interest to the Profession. There is little which can be added to Mr. Rands' statements, and the writer thoroughly agrees with his conclusions as to this particular structure.

In such a foundation as was found in Estacada it seems that the only reason for making extraordinary efforts to secure an impervious cut-off at great depth would be for the purpose of saving water, and that it was not at all necessary to prevent undermining, as the rate of flow through this foundation was so slow as to be negligible. If, however, some further protection was deemed necessary, the writer believes that the concrete cut-off could have been carried very considerably deeper than was done for a lesser sum than was expended on the grouting, and with much more definite assurance of success. If, in addition to this concrete cut-off, a carpet of fine silt, either of clay or volcanic ash or other fine material—which is present in large quantities—had been sluiced into the river bed and against the dam and extending several hundred feet up stream, it would appear certain that a very satisfactory seal would have been obtained. Indeed, the writer is confident that the natural deposit of silt which is inevitable in such a pond will seal this structure very completely, and in a comparatively short time.

Mr. Cushman. William H. Cushman, M. Am. Soc. C. E. (by letter).—The general scheme had been adopted and some of the earlier drilling accom-

plished at the time the writer took general charge of the project. The execution of this portion of the work, as already determined, became a duty, regardless of an individual lack of confidence in the ultimate successful accomplishment of the end in view. Mr. Cushman.

That the foundation material was sufficiently good to carry all the pressure to which it could possibly be subjected, will be conceded by all engineers familiar with the designs for the dam and the foundation material.

Although the breccia was porous, it was substantially free from open cracks, and it was scarcely to be considered a case where the percolation of water through the foundation material would result in undermining the dam.

Admitting these two propositions to be true, it would seem that the only practical consideration would be the one mentioned on pages 35 and 36* of the paper, that is, whether the water lost by percolation would be valuable enough to justify the cost of the grouting work.

Although the Clackamas is a clear-water stream, undoubtedly some silt depositing could be depended on, and possibly selected materials could have been deposited in the pool above the dam to assist materially in silting up the porous material.

Throughout the paper (as mentioned on page 20, the seepage test detailed on page 22, and again on page 28,* where serious thought of discontinuing grouting was entertained), considerable skepticism is expressed as to the effectiveness of the method of treatment. The quoted opinion of the author, that "Over those parts where it was needed it did little good", will be commonly agreed with, and cause little surprise to those who have had similar problems to solve.

The suggestion mentioned on page 37,* that the rock between the rows of drill holes be shattered with small charges of powder, originated with the writer, and it still seems that such shattering might have facilitated the flow of "thickened grout" and made it possible to restrict the treated area to the immediate vicinity of the cut-off wall, rather than diffuse it to remote points by subjecting it to the heavier pressure required to force it through small openings.

It would be interesting to know whether an attempt has ever been made to shatter a trench through rock, to a depth of as much as 50 ft., and then introduce grout or thin mortar (fed through pipes from the bottom and flowing upward or otherwise) in such manner as to produce, in effect, a concrete wall to the depth stated. It would not seem impracticable to accomplish this, and do away with surface capping with concrete and the use of excessive pressure.

Considerable thought and investigation were given by Messrs. Rippey, Fitzgerald, Schreiber, and the writer, to various methods of cut-

* *Proceedings*, Am. Soc. C. E., for January, 1914.

Mr. Cushman.

ting an open trench or seam through the rock to the required depth, in order that an impervious curtain could be introduced. Special channeling machines, drill holes closely spaced and then broaching out the material between holes, and even open cut, were considered. It would be of interest to hear from engineers who have accomplished something along these or similar lines.

Mr. Rands is to be commended for the painstaking manner in which he has presented this interesting subject, and, though it be admitted that the procedure was, to a large extent, futile, nevertheless, his subsequent work has proven the value of this first experiment.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ERNEST PONTZEN, Cor. M. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1913.

Ernest Pontzen was born at Budapest, Austria-Hungary, on January 20th, 1838. He obtained his early engineering education at the Polytechnic School of Vienna. In 1856, he went to Paris and entered the French engineering school, l'Ecole Nationale des Ponts et Chaussées, as an "Externe" student, and was graduated in 1860 at the head of his class.

After a study trip in England, Mr. Pontzen returned to Austria-Hungary, and filled various positions in connection with the railways and harbors of that country. He was Consulting Engineer to the Anglo-Austrian Bank for 4½ years, and, in that capacity, directed the construction of several railway lines, made a study of various railway projects, and became the Director of a railway company in Hungary.

In 1873 he made his first study trip to the United States. He made a second trip in 1876, to serve as member of the Jury in the Railway Group at the Philadelphia International Exposition. Returning to Paris, he established himself there permanently, and eventually became a naturalized French citizen. He married Miss Hirtz, daughter of the distinguished physician and professor of that name, of the University of Strassburg, who survives him.

Mr. Pontzen was the author of numerous publications dealing with the construction and operation of railways. His most important work, issued in collaboration with Mr. Lavoinne, and entitled "American Railways," was published in 1880-82, the first volume dealing with the construction and cost of railways, and the second with their operation.

Mr. Pontzen served on various important commissions. He had been a member of the Government Commission for the technical exploitation of railways since 1884, almost from its inception. Acting in this capacity, he was one of the delegation which represented the French Government at the International Conference for the promotion of technical uniformity on railways, and more recently he was a delegate of the Minister of Public Works to the International Railway Congress at Washington, and, later, to that at Berne.

* Memoir prepared by Charles L. Strobel, M. Am. Soc. C. E.

He was also selected as an expert and arbitrator in very important cases, such as the Loetschberg Tunnel, the Harbor of Varna, the acquisition of the Algerian East, and the Cé Bridge accident.

He was a Member of the Council on Instruction (Conseil d'Instruction), and of the Council on Betterment (Conseil de Perfectionnement) of l'Ecole Nationale des Ponts et Chaussées.

Mr. Pontzen was a Past-President of the Society of Colonial Engineers and Past Vice-President of the Society of Civil Engineers of France. He was one of the founders and, up to the time of his death, was President of the Friendly Society of "Externe" Graduates of l'Ecole Nationale des Ponts et Chaussées. He was also an officer of the Legion of Honor and a Commander of various French and foreign orders.

This brief outline of Mr. Pontzen's career will indicate that it was fruitful in engineering achievement and rich in well-merited honors. He went to France as a stranger, and died in the country of his adoption, having won not only the esteem of Frenchmen, but to a singularly high degree their love and affection.

Mr. Pontzen spoke French, German, English, Italian, and Hungarian, and with foreign engineers whom he met at Congresses, he was able, therefore, to converse in their mother-tongue. All who met him in this way remember him most pleasantly for his uniform courtesy and cheer. American engineers who had the good fortune to know him were always sure of a cordial welcome at his hands, and he freely gave them of his time and attention in the promotion of their interests or their comfort and pleasure. He was the kindest, most considerate, and most devoted of friends. It may be said of him that the conduct of his life was so noble and inspiring, that the highest tribute will be paid to it that can come to any man, in that it has left the desire, especially on the part of the younger men, to emulate his example as best they can.

Mr. Pontzen was elected a Corresponding Member of the American Society of Civil Engineers on January 5th, 1876.

ARTHUR LINCOLN ADAMS, M. Am. Soc. C. E.*

DIED SEPTEMBER 17TH, 1913.

Arthur Lincoln Adams, son of Jacob Clendennin and Nancy McCoy (Hamilton) Adams, was born on September 15th, 1864, on a farm near Greensburg, Ind. He was a lineal descendant of John Adams, who came to Plymouth Colony, Massachusetts, in the ship, *Fortune*, in 1621.

* Memoir prepared by Franklin Riffle and W. A. Cattell, Members, Am. Soc. C. E., and A. Kempkey, Assoc. M. Am. Soc. C. E.

When about eight years of age he moved with his parents to Topeka, Kans., where the family resided for a period of about three years, and then returned to Indiana. In 1882, soon after the death of Mr. Adams' father, the widow again moved with her children to Topeka, where she resided during the remainder of her life.

In his early boyhood Mr. Adams attended the district schools of Indiana, and the public schools of Topeka. Later, he entered the Greensburg High School, from which he was graduated in 1882. He was a student at Hanover College, Ind., in 1882-83, and Washburn College, Topeka, Kans., in 1883-84.

During the summer of 1883 he was in the employ of the County Surveyor of Shawnee County, Kansas, and assisted in surveying in and about Topeka. During the following summer he was with a surveying party engaged in subdividing Government land in Southwestern Kansas. Starting as a Chainman, he was promoted to Transitman before the close of his engagement. When the survey was completed, each member of the party was requested to state, under oath, that the work had been done correctly, to the best of his knowledge and belief. Being somewhat in doubt as to the accuracy of the survey, Mr. Adams, together with several others of the party, strenuously objected to being sworn. At this juncture, one of the youthful mutineers made the discovery that Quakers were permitted to affirm in lieu of taking oath, whereupon Adams insisted that the same privilege be accorded him and his conscientious associates. His arguments prevailed, the affirmations were duly made, and all the members of the party were given honorable discharges.

In the fall of 1883 students of the State University of Kansas started a movement to organize a State oratorical association, to affiliate with the Interstate Association. Mr. Adams was a delegate from Washburn College to the constitutional convention. He took a prominent part in the proceedings, which resulted in the organization of a State association, and was active in arranging for the first contest. He participated in the trial and settlement of a famous case of plagiarism which developed in that contest.

In 1884 he became a student in the Engineering Department of Kansas State University, and was graduated in 1886. At a public rhetorical recital he read a humorous essay entitled "The Philosophy of Selfishness", in which his theories and conclusions were so extreme as to create a sensation among his hearers. Only a few of the audience appreciated the undercurrent of humor which pervaded the essay, or realized that the extreme views advocated by the speaker were purely for rhetorical effect.

During the spring of 1886, he obtained employment in the Engineering Department of the Burlington and Missouri River Railroad, in Nebraska, in the office of one of the resident engineers, and was soon

promoted to Division Engineer on Construction. In this position he displayed unusual efficiency, originality, and judgment, for one of his age and experience.

During the summer of 1887, he concluded to try his fortune on the Pacific Coast. He accordingly accepted a position as Leveler in a location party on the Oregon Pacific Railroad. When the party was disbanded, during the following winter, he went to Southern California in search of employment and spent several months in Los Angeles.

During the spring of 1888 he returned to the Pacific Northwest to accept a position as Assistant Engineer on Construction, offered him by the Chief Engineer of the Oregon and Washington Territory Railway, now a part of the Northern Pacific System. In this position he had an excellent opportunity to observe and study the wonderful resources of Eastern Oregon and Washington. He was quick to grasp the significance of the development of this vast inland empire, and to see with prophetic eye the varied and multitudinous engineering problems to which this development would give rise. Many thriving towns were in various stages of development, and some of them had already reached a point where the necessity for sewer and water systems was occupying the attention of their citizens.

Up to this time, Mr. Adams, like many other young engineers, had followed the lines of least resistance. He had become a railroad engineer through force of circumstances. After graduation he had accepted a position on a railroad because it was the only position immediately available. Municipal engineering, however, was his choice, and he now determined to make a start in this field. The Town of Pendleton, Ore., had recently created the office of "City" Engineer, and the Mayor was given the appointing power. Before resigning his position as railroad engineer, Mr. Adams discussed the matter with the Chief Engineer, who accepted his resignation with regret, but, nevertheless, co-operated with him in his efforts to secure the coveted municipal position. Mr. Adams was successful in obtaining the appointment, at the same salary he had received from the railroad. In conducting the negotiations which terminated so happily, he exhibited that remarkable business sagacity which contributed so largely to his later successes. Foreseeing that as "City" Engineer of a small town his duties would not be very onerous, he stipulated that he should be granted the privilege of performing such private work as he might see fit to undertake during his term of office. He prepared a contract fully covering this phase of the question, and it was duly signed by the contracting parties. It was not long, however, before his private practice assumed such large proportions that he found it advisable to resign.

In 1890 Mr. Adams prepared plans and specifications for a system of water-works for the Town of Dayton, Wash., and superintended its

construction. During the following year he performed a similar service for Colfax, Wash.

In the summer of 1891 he entered into partnership with his brother-in-law, Robert C. Gemmell, M. Am. Soc. C. E., under the firm name of Adams and Gemmell. The new firm designed and superintended the construction of water-works for La Grande, Ore., and Waitsburg, Wash. They also designed a water system for Heppner, Ore., and a water-power plant at Kalama, Wash., neither of which was constructed, however, owing to the financial panic of 1893. The firm did considerable miscellaneous surveying in Oregon and Washington, including two large irrigation canals on the Umatilla River, Oregon, one on the Walla Walla River, Washington, and one on Eagle Creek, near Baker City, Ore. The construction of these canals was indefinitely postponed by the "panic". They prepared plans and specifications for a water-works system for Athena, Ore., and a sewer system for Montesano, Wash., both of which were constructed, although not under their supervision.

During the fall of 1893 Mr. Adams made a very complete report on an increased water supply for Astoria, Ore., and recommended the construction of a new system. His report was adopted in its entirety by the Board of Water Commissioners, and Adams and Gemmell were authorized to make the necessary surveys and plans. Before the completion of the preliminary surveys, the partnership was dissolved, and the work was continued by Mr. Adams, who, as Chief Engineer, carried it through to a successful termination—less than 3 years after having made his preliminary report. An admirable description of this work was written by Mr. Adams, under the title, "The Astoria City Water Works".* This paper won for its author the Thomas Fitch Rowland Prize for 1897. In his paper he described quite fully an ingenious device (of his own invention) for opening and closing two fire gates from a central station; this being accomplished "by opening and closing an ordinary stop-cock at the end of a line of ordinary ¾-in. service pipe leading to a patented governor attached to each of the gates."

During the construction of the Astoria Water-Works (1895), Mr. Adams prepared a report on a new water system for Eureka, Cal., and on the character and probable cost of the private works then supplying the city. After the completion of the Astoria Water-Works, he opened a consulting office in San Francisco. During 1897 he prepared plans and specifications for the reconstruction of water-works at Centralia, Wash., and also reported on projected new works for Lakeport and Oceanside, Cal.

From 1897 to 1900, he held the dual position of Manager and Chief Engineer of the West Los Angeles Water Company and the West Side Water Company. During this period he resided in Los Angeles, where

* *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 1.

he also carried on a consulting practice. As Chief Engineer of the two Los Angeles Water Companies, he was called on to plan and construct extensions in order to keep pace with the steadily increasing demand. He constructed a 5 000 000-gal. reservoir, and a pipe line to the Soldiers' Home, Santa Monica. In connection with the pipe line, he designed an ingenious weir and gate device for maintaining a definite delivery under varying conditions of pressure.

During this period Mr. Adams filled various engagements, among which were the following:

He was a member of a commission of engineers engaged by the City of Los Angeles to determine the value of the improvements made to the water-works of that city during the 30 years' lease by the Los Angeles City Water Company; he was a member of a commission of engineers engaged by the City of Pasadena to appraise the several water-works used for the supply of that city; in conjunction with J. B. Lippincott, M. Am. Soc. C. E., he reported on sources of supply and plans for an entirely independent system of water-works for Pasadena; he developed a water supply and prepared plans for its utilization, in the irrigation of a large ranch near Corona, Cal.; he prepared a report outlining plans, and giving estimates of cost, for a combined municipal water supply and power installation for Missoula, Mont.; he prepared plans and specifications for the reconstruction of the water-works of Pendleton, Ore.; he reported on the probable effect on the water supplies of the San Gabriel River, California, of the contemplated construction of certain power plants on that stream; he prepared a report on the value of the properties of the West Los Angeles Water Company and the West Side Water Company, as an aid in effecting a sale of these properties to the City of Los Angeles; he prepared a report and gave evidence concerning the value of the properties of the Contra Costa Water Company used for supplying the City of Oakland and its vicinity; and he prepared a report and gave evidence concerning the causes of an extensive land-slide which had for some years prevented the use of the large reservoir on the west side of the Willamette River, at Portland, Ore.

In 1901 Mr. Adams became Engineer and Manager of the Contra Costa Water Company, Oakland, after having been retained as an expert in the important litigation case which had just at this time been concluded.

The case was the now famous one known as the Hart case, and was instituted to enjoin the City of Oakland from enforcing the rate ordinance enacted by the City Council. Although several other eminent engineers were engaged in the case, it was conceded by all that the clear and convincing testimony of Mr. Adams as to values, and his clear exposition of the theories of valuation, had more to do with the victory of the Company than any other one thing. The Company

was granted in effect an increase in rates amounting to 15%, and, although in later years engaged in many important legal matters, Mr. Adams often remarked that he considered this case the most important of its kind in his career.

Early in 1903, Mr. Adams resigned from the management of the Contra Costa Water Company and opened a consulting office in San Francisco, and from this time he continued steadily his consulting work. Although his practice was never such as to require a large organization, it was at all times of the highest type. Among the more important of his engagements may be cited that with the City of Victoria, B. C. His report dealt with the existing conditions and present and future needs, covering the subject in complete and exhaustive detail. Later, he prepared plans and specifications covering the then present needs, and, under his direction as Consulting Engineer, they were executed.

While still retaining his consulting practice, he was, in 1906, made Chief Engineer of the Department of Greater Water Supply of the Peoples Water Company, successors to the Contra Costa Water Company, supplying water to seven cities on the east side of San Francisco Bay. Complete preliminary studies and investigations were carried on by this organization looking to the more than doubling of the water supply of this Company. A complete comprehensive plan of future development was outlined, and construction was begun on the more important elements, but financial stringency prevented carrying out more than a small portion of this work.

Mr. Adams did a very large amount of valuation work, particularly of public utilities. Among the more important properties on which be reported may be mentioned the Peoples Water Company, the Spring Valley Water Company, the Palermo Land and Water Company, the Virginia and Gold Hill Water Company, the Petaluma Power and Water Company, and numerous others.

In 1909, he designed the irrigation system for the Patterson Ranch Company, at Patterson, Stanislaus County, California. This system provides a means for irrigating, by pumping, about 18 000 acres of land on the west side of the San Joaquin River in the San Joaquin Valley, and was at that time considered to be of unique design and the most extensive and complete system of the kind in the United States.

At the time of his death Mr. Adams was engaged in the construction of a plant similar in design to the Patterson Ranch system, but more elaborate as to details, for the irrigation of about 22 000 acres of land near Brentwood, Contra Costa County, California. He was also, at the time of his death, Consulting Engineer for the Peoples Water Company, and was engaged by this Company in important rate litigation pending in the Federal Court.

Mr. Adams was foremost among those who organized the San

Francisco Association of Members of the American Society of Civil Engineers. He was elected Treasurer of the Association for a term of 2 years, and became its third President in 1907. Immediately after the earthquake in 1906, he was instrumental in holding a series of special meetings of the Association, which resulted in a study of "The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions," by a general committee and six special committees composed of members of the Association, followed by extensive reports.* Mr. Adams was Chairman of the Committee on the Effects of the Earthquake on Water-Works Structures, and wrote the Committee's report.

Mr. Adams held a high rank both personally and professionally. He was exceedingly democratic in his ideas, very simple in his tastes, and exceptionally companionable. He possessed a keen, analytical mind that served him well in the litigation with which he was so frequently connected as an expert witness. He delighted in the legal subtleties of these cases no less than in the engineering problems involved. Although active in assisting and supporting the attorneys with whom he was associated, he always commanded the admiration and respect of the engineers and attorneys who opposed him. His ability was acknowledged and his work commended by friend and foe alike. His professional career was one of uninterrupted achievement from its beginning to its untimely end. Long before his death he had attained a foremost place in the ranks of the leading hydraulic engineers of the Pacific Coast. He possessed in a marked degree the rare combination of professional skill and business acumen which enabled him to procure for his clients the best attainable results at a minimum cost. Having early in his career established a reputation for efficiency and integrity, he thereby created a demand for his services, for which, usually, he was able to name his terms. While always fully alive to his personal interests, he never for a moment overlooked the interests of his profession. He continuously sought to uphold its dignity, and to educate the public to a proper conception of its importance as a factor in the world's development. On numerous occasions he fearlessly exacted from his clients the recognition to which he believed his profession was entitled.

Mindful of his early struggles for recognition, he strongly sympathized with young engineers, and was ever ready to extend a helping hand to those in need of assistance. It was largely the desire to aid the younger members of the profession that induced him to take a prominent part in the organization of the San Francisco Association of Members of the American Society of Civil Engineers. He believed that they, more than the older members of the Society, would be benefited by membership in this organization. During the practice of his

* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 208.

profession he gave employment to many young engineers. He invariably treated his subordinates with forbearance and tolerance, and thus secured and retained their respect, loyalty, and friendship.

Mr. Adams possessed a gift which, unfortunately, is quite rare among engineers—that of clear, terse, forceful expression. This applied to his speech no less than to his writing. Whether writing a report, explaining an engineering proposition to a client, elucidating a technical point while on the witness stand, or addressing an audience, he invariably chose his words well, and without hesitation. In argument he was convincing and rarely failed to carry his point. Whenever he took a stand for or against a proposition, he gave the impression of being master of the situation. He never allowed himself to be swerved from the course which his conscience or his judgment marked out.

His energy and ability, however, were not devoted entirely to his engineering practice. He was a zealous and efficient worker in the cause of religion and morality. He was prominent in the affairs of the church of which he was a member, and also gave much of his time and resources to the Young Men's Christian Association and Young Women's Christian Association of Oakland. The Y. M. C. A. Building, and the new First Presbyterian Church edifice, both in Oakland, are to a very large extent the results of his effective work.

Mr. Adams was a well-read man. He was especially fond of poetry. Among the poets, Robert Burns was his favorite. He occasionally wrote verse of a serious nature, although not for publication. One of the writers of this memoir has in his possession several of Mr. Adams' poems, one of which, entitled "A New Year's Revery", possesses exceptional merit.

Frank T. Oakley, M. Am. Soc. C. E., who was a classmate of Mr. Adams in the public schools of Topeka and in the State University of Kansas, and also was associated with him during his professional career, writes of his life-long friend as follows:

"He was always very tolerant of the beliefs of other people, never seeming to wish to force his opinions upon others. However, when it came to argument, few could hold their own against him. My observation leads me to believe that he always followed a thoroughly digested plan of conduct and business from which he did not allow himself to vary in principle, and I believe much of his success in life's problems was due to this fact as well as to his unusually good judgment.

"Some time over twenty years ago, while sitting in camp, he outlined to me his idea of an engineering career leading up to a consulting business. When I came to live in California, I was impressed with the similarity of his outline of a career and his experience, and the way he carried on his professional business. He always insisted on giving any undertaking his best thought, and would not be identified

with it otherwise, not sparing himself hard work; and as a consequence he was always accorded a leader's place by his associates.

"I believe that Arthur L. Adams was the finest character I have ever known, and that it is not possible for me to know how much I owe to association with him. His influence was always for good and manly things. This feature did not eliminate, or in any way curb the mischievous or boyish spirit, or the joy in sport or in a practical joke, but it did eliminate any meanness—a thing that was impossible in his nature.

"In our close association, including two years of living together, I never knew him to be gushingly confidential. There was always something of a reserve about him. Nevertheless, I never knew one in whom I could more freely confide, and who was more ready with the best and most kindly and tactful advice. I knew him best during the character-forming age, when boys are fully themselves and freely open to their playmates. Yet I never knew him to do an act or express a thought that was not in accord with his later high moral and mental attainments. Nor was he in the least prudish, nor did his associates ever think of calling him a 'goody-goody boy.'

"During our college days we frequently discussed all kinds of subjects pertaining to human life and affairs. Whether the subject was a social matter, the mathematics of gambling, or theology, freedom and frankness of thought and expression were always the same, always tending to a greater and more beneficial knowledge of the subject and, in fact, a crystallization of one's own opinions. When thinking of Arthur Adams' character, I am reminded of what Riley makes the farmer say of his friend and neighbor:

> "'Fer the name of William Leachman and
> True Manhood's jist the same.'"

C. Derleth, Jr., M. Am. Soc. C. E., pays the following tribute to Mr. Adams, both as a man among men and an engineer:

"Through his writings, particularly on wooden stave pipe, I have known Mr. Adams by reputation at least since 1896. I did not meet him personally until 1904. But since then I feel that I have had an opportunity to know of his work and to appreciate his personality. For the last eighteen years I have worked for, or have met in various ways, a great many engineers whose names are prominent in our national societies, and these men hail from all quarters of the United States. I am convinced that among them none has combined in the highest sense, better than Mr. Adams did, all of those qualities which contribute to the professional and personal character of an engineering gentleman. One always associated with Mr. Adams the qualities of integrity, industry, wide reading, learning in fields other than engineering. He had a great command of the powers of speech. He wrote fluently. He was an engineer who used reason and who had powerful judgment. Above all, he was a man of the finest moral instincts. He was always ready to help improve the tone of engineers and to raise the standard of engineering ethics. Indeed, he applied the same qualities to wider fields and was a power for good and uplift in the community in which he lived. As more men like Mr. Adams

become identified with engineering, so in proportion will the community's respect for the profession of engineering increase."

Augustus Kempkey, Assoc. M. Am. Soc. C. E., writes of Mr. Adams as follows:

"As his Principal Assistant for some eight years immediately preceding his death, the writer came to know Mr. Adams as a most refined and courteous gentleman of the highest type, every ready to give the best that was in him to the uplift of humanity and to the assistance of those around him. His thoughts were ever on the highest plane and his judgment, particularly in engineering matters, remarkable to a degree that at times seemed little short of uncanny. His mind, working perfectly clear and along direct lines, enabled him to grasp the essentials of an engineering problem almost at once, and his solution of these essentials was equally direct and expeditious. His criticisms of and orders to his subordinates were always given in a most kind and considerate manner, and no matter what the subject, he was always ready to receive suggestions and ideas from those directly under him. In all my association with him, I never saw him either reject or accept a suggestion without being fully informed as to its significance and value; and in the case of rejection, pointing out clearly his reasons therefor, and if accepting, giving due praise and weight to the value thereof. In short, he was a man who inspired and encouraged ideas and suggestions from his subordinates, while withal he never provoked that familiarity which ultimately breeds contempt. In each of his reports it will be found that he never failed to give credit to those who had assisted him in their preparation.

"He believed the engineering profession to be one of the greatest, and maintained an ethical standard such as few can hope to measure up to. By the life and conduct of such men as he, the engineering profession will be raised to its highest plane, and the moral tone of all with whom they come into contact cannot fail to be enriched. With all who had the privilege of working with Mr. Adams, and with all who came into contact with him in any walk of life, his memory will live forever as that of a true gentleman and an eminent—or rather a pre-eminent—engineer."

Mr. Adams was married on December 18th, 1889, to Mary Gemmell, of Topeka, Kans., who, with two sons and three daughters, survives him. Robert, the eldest, is a student in the Engineering Department of Stanford University, and two of the daughters are students in the University of California.

Mr. Adams died at his home in Oakland, Cal., on September 17th, 1913, of pneumonia, after an illness of less than one week. In his death the Engineering Profession has lost one of its great leaders; the community an active and efficient instrument for civic and social betterment; and his family and friends a wise counselor and a genial, lovable companion.

Mr. Adams was elected a member of the American Society of Civil Engineers on October 2d, 1895. From 1907 to 1909 he served as a

Director. He was President of the San Francisco Association of Members of the American Society of Civil Engineers during 1907. He was a member of the Franklin Institute, the American Academy of Political and Social Science, the Technical Society of the Pacific Coast, the Pacific Association of Consulting Engineers, and of the First Presbyterian Church of Oakland. He held important offices in his church, and was Vice-President and Director of the Young Men's Christian Association of Oakland. He was also a member of the Delta Tau Delta and Sigma Xi college fraternities.

RICARDO MANUEL ARANGO, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1914.

Ricardo Manuel Arango, Consulting Engineer for the Republic of Panama, died at his home in the City of Panama, on January 24th, 1914, after a long illness. He was born on December 25th, 1864, and was graduated in 1887, with the degree of C. E., from the Rensselaer Polytechnic Institute, at Troy, N. Y. His first work after graduation was a survey, under the direction of the late Pedro J. Sosa, M. Am. Soc. C. E., a prominent engineer of Panama, of 500 000 hectares of land in the Province of Bocas del Toro, conceded by the Colombian Government to the French Canal Company. This party did the first surveying work in that region.

In 1889, Mr. Arango was appointed by the Colombian Government to take charge of surveys made necessary by a controversy between it and the Panama Railroad, over the filling of the present site of Colon. In 1890, he was engaged on the railroad which was built to haul to the coast the products of the manganese mines of Viento Frio, Province of Colon. He assisted in the surveys and plans for providing the City of Panama with water from the Juan Diaz River, but this development was never realized.

Mr. Arango took an active part in the Revolution of 1903 by which Panama secured its independence from Colombia, being of great assistance to his father, José A. Arango, who was a member of the original Junta of Separation. As member of the Municipal Council of the City of Panama, Mr. Arango signed the Act of Independence, and was appointed the first Chief Engineer of the new Republic.

Under John F. Wallace, Past-President, Am. Soc. C. E., Chief Engineer of the Isthmian Canal Commission, Mr. Arango was Consulting Engineer for sanitary work in the City of Panama. Later, he was made Division Engineer of the Bureau of Meteorology and

*Memoir prepared by Alex. P. Crary, Assoc. M. Am. Soc. C. E.

River Hydraulics. As head of this Bureau, he installed the first seismograph on the Isthmus, and established the gauging station at Alhajuela, on the Chagres River, from which advices of any floods could be telephoned in advance to those in charge of work on the lower reaches of the river. His services with the Isthmian Canal Commission were terminated in the fall of 1908. Soon afterward Mr. Arango was appointed Minister Plenipotentiary and Envoy Extraordinary for the Republic of Panama to the Court of St. James, which post he was compelled to relinquish in 1909 on account of ill health. In 1910, he was again made Chief Engineer of the Republic, and, later, Consulting Engineer, which position he held at the time of his death.

He was a Member of the Instituto de Ingenieros de Chile, the Seismological Society of America, and the Sociedad de Ingenieros, Arquitectos y Agrimensores de Panama, of which latter he was the founder. On September 21st, 1899, he was married to Miss Maria Lewis, who, with five children, survives him.

Mr. Arango was a man who loved his profession. Even during his long illness, when he had lost almost completely the use of his hands, he took great delight in reading and studying engineering. In a wonderful manner he had trained his mind so that he could transform equations, construct graphical diagrams, and do things mentally which are generally accomplished by paper and pencil. He was always genial and willing to help and give advice. He stood up courageously under his sickness though it deprived him of the many pleasures and much work which make life worth living. He was always interested in public affairs, and, as long as he could, he took an active part in them. In his death, Panama loses one of its best and most progressive citizens.

Mr. Arango was elected an Associate Member of the American Society of Civil Engineers on September 2d, 1896, and a Member on February 6th, 1906.

JOHN WILLIS HAYS, M. Am. Soc. C. E.*

DIED DECEMBER 14TH, 1913.

John Willis Hays was born at Oxford, N. C., on March 14th, 1861, and was christened with the name borne by his father and his grandfather. He was educated at local private schools and the University of North Carolina.

As a young man Mr. Hays did engineering work under the late Benjamin D. Frost, M. Am. Soc. C. E., of Massachusetts, who had distinguished himself in connection with the building of the Hoosac Tunnel in that State, and was, at the time in question, engaged in laying out a route for the Oxford and Henderson Railroad. Later,

*Memoir prepared by Francis B. Hays, Esq.

Mr. Hays joined the field staff of William C. Kerr, State Geologist of North Carolina, where he acquired experience and developed ability which gained for him an appointment as a Topographer in the United States Geological Survey. This position he held for many years, first in the Appalachian Mountains, and later in the Rockies, during the summer, and at Washington in winter.

In addition to the remarkable thoroughness of his engineering work, Mr. Hays was distinguished for the unusual excellence of his drafting. Indeed, his artistic ability was marked, and had he chosen Art as a vocation, his success would doubtless have been gratifying. Nor was his facility with the pen confined to drafting and free-hand sketching, for many of his exciting mountain adventures were reduced to manuscript and published in such periodicals as *The Outing Magazine, The Youth's Companion,* and the newspapers of Washington and other cities. He was also an occasional contributor to journals devoted to engineering matters.

Leaving the Government service in the early Nineties for work in which his individuality would have freer play, Mr. Hays became City Engineer of Petersburg, Va., having a few years previously married the daughter of one of the leading citizens of that place, who, with a large family, survives him. While in that position he laid out Ferndale Park, constructed the high-service reservoir, and in other ways left his impress on the city's physical characteristics.

Mr. Hays' nature was such that he could not be contented in a political position. He knew of but one course to pursue, and that was the one of absolute rectitude as he saw it, and he was remarkably free from moral strabismus. Consequently, he left the public position to engage in private work. Although frequently urged to return to the City Engineership at an increased salary, he steadfastly and politely declined.

In the meantime, he built up a private practice which became more remunerative as the years went by, and he was able to command the fee of an expert for a mere opinion. Shortly before his death, he had completed the development of the Walnut Hills Section, reached from the City of Petersburg proper by an iron viaduct over a broad gorge which latter had long rendered it practically inaccessible. He developed the water power at Roanoke Rapids, N. C., and did municipal work at Mt. Airy, Dunn, Kingston, and other towns in his native State, as well as at Emporia, Blackstone, and many other points in the State of his adoption.

Physically, Mr. Hays was a man of magnificent mould, having been considerably more than 6 ft. in height, erect and clean-limbed, and carrying his two hundred and odd pounds without the suggestion of a surplus ounce of flesh. He was cut down at the zenith of his powers by thrombosis induced by a fall sustained while engaged in field work.

Mr. Hays was Master of Blandford Lodge of Masons, President of the Petersburg Benevolent Mechanics' Association, an Elk, and, for several years, had conducted with marked success a men's Sunday-school class. He was a man of vigorous mind, keen intellect, catholic tastes, strong personality, a faithful friend, and a true gentleman.

Mr. Hays was elected a Member of the American Society of Civil Engineers on June 5th, 1901.

LUTHER REESE ZOLLINGER, M. Am. Soc. C. E.*

DIED OCTOBER 21ST, 1913.

Luther Reese Zollinger, the fifth son of William George and Susannah (Spece) Zollinger, was born on April 25th, 1865, in Harrisburg, Pa. He received his early education in the public schools of his native city, having been graduated from the High School in 1883. Of his early life his most intimate boyhood acquaintance, Dr. C. R. Phillips, of Harrisburg, writes:

"No member of our class in High School, where we were associated for four years, held his classmates by the mere strength of a lovable personality as did Luther Reese Zollinger. We were all devoted to him because of that indescribable something he had, which, unfortunately, is uncommon, and made us certain of his sincerity of purpose and life. He was honor man in our class in High School, easily so too, and I have always felt that he was honor man in our class at Lehigh, though it is true that one or two others had a few per cent. higher standing, according to the poor, insufficient ratings which, as things educational stand, must still be taken to measure a man's mind and ability. Of all the men whom I have known, he had, I always felt, the best mind for mathematical work. In saying this I, of course, except a few neurotics whose mathematical ability always is a part of an unstable mental equilibrium."

In 1884, Mr. Zollinger entered Lehigh University, and was graduated in 1888 with the degree of Civil Engineer. He was President of his class in the Sophomore year, Editor of the College *Epitome*, and Business Manager and Editor of the *Engineering Journal*, and had a Commencement appointment. During his 4 years in college he never ranked lower than fifth, and for the last 3 years he ranked second in a class varying from 101 to 66 men.

Mr. Zollinger's entire professional career was spent in the service of the Pennsylvania Railroad Company, in which he rose by his own efforts and merit from the humble post of Chainman to the position of trust and responsibility which he occupied at the time of his death. Starting in the office of the Assistant Engineer, Middle Division, at Harrisburg, Pa., on March 11th, 1889, he received his first professional

* Memoir prepared by J. F. Murray and C. J. Parker, Members, Am. Soc. C. E.

experience at the time of the great floods throughout Pennsylvania, in May and June of that year, which proved so destructive to the railroads and other internal improvements in various parts of the State, especially in the vicinity of Johnstown, and along the Susquehanna, Juniata, and Connemaugh Rivers.

He held successively, and for varying periods from 1889 to 1905, the various positions of Transitman in the office of the Engineer of Maintenance of Way, Pennsylvania Railroad Division, at Altoona; Assistant to Assistant Engineer, Philadelphia Division, at West Philadelphia; Assistant Supervisor, Division No. 8, Middle Division, Pennsylvania Railroad, at Spruce Creek; Supervisor, Division No. 28, at Norristown; Supervisor, Division No. 1, Philadelphia Division, at West Philadelphia; Assistant to Principal Assistant Engineer, and Principal Assistant Engineer, Pennsylvania Railroad Division, at Altoona.

On April 1st, 1905, Mr. Zollinger was promoted to the position of Engineer of Maintenance of Way, General Office, Philadelphia, which position he held at the time of his death. In this capacity he was in charge and had direct control of the Maintenance of Way Department, in so far as was necessary to insure the efficiency of the Department and adherence to the standards of the Company. He also had charge of the preparation of all maintenance of way plans, the issuing of instructions in regard to adherence to the same, and personally examined all bridges and other structures, reporting on their condition and making recommendations in connection therewith, as well as with respect to other matters relating to maintenance of way.

In addition to these duties, Mr. Zollinger, in 1905-07, was in charge of the construction of extensive yards and yard facilities at Morrisville on the New York Division, and at Shire Oaks on the Monongahela Division; he was also appointed as Chairman of various committees, by the General Manager, to make experiments, investigations, and reports on sundry matters relative to the economical construction and maintenance of track. One of these reports,* dated March, 1911, covers extended experiments to determine the necessary depth of stone ballast. Of this report, W. C. Cushing, M. Am. Soc. C. E., Chief Engineer, Maintenance of Way, Pennsylvania Lines West, says: "These tests are the most extensive of the kind ever conducted in this country, and will be found of great interest and value to railway engineers."

Mr. Zollinger traveled extensively, both in this country and in Europe, and his wonderful powers of observation, retentive memory, quick comprehension, enthusiasm, and capacity for work, made him invaluable to the company he served. As an example of his enthusiasm and wonderful resourcefulness, he purchased a tract of land at

* *Proceedings*, American Railway Association, Vol. 18.

Merion, Pa., on which was an old, abandoned stable, and with the aid of his architect transformed it into a beautiful home of the English manor type,* utilizing for the interior finish and decoration the materials from an old mansion formerly on the estate.

His personality was peculiarly lovable and attractive, and left its impress deep on all who met him. One felt immediately that he was a man worth knowing, an impression that association only strengthened and confirmed. Big of body, handsome of feature, with great magnetism, his winning personality drew all classes of men to him, and he held them with hooks of steel by his genial, broad-minded, and tolerant disposition. He was by nature a man of essentially social tendencies. Of fine intellectual gifts, broad education, wide acquaintance with the best literature, discriminating taste in the fine arts, varied experience with many classes of men, an observant traveler, with a retentive memory, great fund of anecdote and reminiscence, and a keen sense of humor, he was always entertaining in conversation. These qualities, with his loyalty, kindness, generosity, and buoyance of spirits, made him a most interesting and delightful companion.

Full of sympathy, he was never too busy to give time to the troubles of his friends or counsel to the youth seeking opportunity. It is not often that a man who has attained high professional standing shows a live interest in the young man just beginning his life work, but Mr. Zollinger was the champion of youth, with an unbounded faith in its capabilities and the fulfilment of its aspirations. This faith he exemplified in a very striking and practical way, for he took more than one young man into his home, gave them his friendship and confidence, inspired them by his manliness, educated them at his own expense, and launched them on honorable and useful careers in life. It was a great satisfaction and source of much pride to him to know that each of these young men fully justified the trust placed in him.

His genial nature, overflowing with good fellowship, so attractive to his many friends, was carried into his domestic circle and was one of the finest phases of his life, as seen by his intimate friends. He never married, and his devotion to his sister can only be described as beautiful. Nothing he did for her was ever felt to be a sacrifice. His happiness was only reached through her happiness. Such affection was warmly reciprocated, and his home was made to him a place of charm as well as of rest.

Mr. Zollinger's dominant characteristic was courageous and vigorous manhood. His power of mind and body was shown in his capacity for concentration and the accomplishment of work. He was of quick discernment and swift comprehension, and his strength made him positive in his convictions and strenuous in maintaining them. With

*Described by the architect in *House and Garden*, December, 1912.

his strong character and self-respect, he could never play the courtier, and he detested to receive the flattery he would never apply to others, not even in the form of studied deference which, at times, might have gained him personal advantage.

Enthusiasm in regard to whatever matter he took up, either work or recreation, was another feature of his character, and made him a natural leader. He was a lover of the beautiful in Nature and art. Nature in all her forms appealed strongly to him. Some of his happiest hours were spent in roaming among the trees of the forest or the flowers of his own garden, whose characteristics and history he knew so well. He kept in touch with the advancement in science and all questions of the day, and could enjoy intelligently and appreciate intercourse with men versed in many forms of learning.

Men of great professional attainments and mental power are often deficient in those qualities of the heart which make them loved as well as honored, but these qualities Mr. Zollinger possessed in an unusual degree. He had intellectual superiority, and he was a man of honorable achievement, but those who knew him well think of him and love him for his broad-gauged, liberal-minded, vigorous manhood and spotless integrity. He was chivalric in thought and deed, and the courteous gentleman always. Honor, truth, and duty were the stars by which he steered his course, and in him was embodied the highest type of manhood. Of splendid physique, clean and wholesome morals, a fascinating personality with a highly cultivated mind, he died in the prime of brilliant achievements. The exquisite charm of his friendship can be known only to those who enjoyed it; to them it was a benediction, and his death an irreparable loss. In a generation we shall not see his like again.

Mr. Zollinger had complained of ill health for several months prior to his death, though he did not relinquish any of his work, and had just returned from the annual track inspection when he was stricken with apoplexy, on the morning of October 16th, and died at his home in Merion, Pa., on October 21st, 1913.

He was a Member of the American Railway Engineering Association and the Delta Upsilon Fraternity.

Mr. Zollinger was elected a Member of the American Society of Civil Engineers on March 6th, 1901.

MURRAY FORBES, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1913.

Murray Forbes was born in Philadelphia, Pa., on June 23d, 1863. His father, Dr. William S. Forbes, was Professor of Surgery at Jef-

* Memoir prepared by W. C. Hawley and G. W. Hutchinson, Members, M. Am. Soc. C. E.

ferson Medical College. His mother Celinere (Sims) Forbes, was a sister of J. C. Sims, Secretary of the Pennsylvania Railroad Company, and Judge Clifford Sims of the Superior Court of New Jersey.

Mr. Forbes received his early education at Rugby Academy and at Dr. Farries' School, in Philadelphia, and entered the Art School of the University of Pennsylvania at the age of fourteen. When he was 17, he entered the Pennsylvania Railroad shops at Altoona, Pa., where he took the regular 4 years' apprenticeship course. He was then employed in the shops for some months, being afterward transferred to Derry, Pa., where for 4 years he was Assistant Road Foreman of Engines on the Pittsburgh Division of the Pennsylvania Railroad.

He resigned this position in 1888 and went to Greensburg, Pa., where he took charge of the construction of the plant of the Westmoreland Water Company. Mr. Forbes continued as the executive head of this Corporation until his death. Under his direction, the plant was extended and, with allied corporations, served the communities along the Main Line of the Pennsylvania Railroad west from Greensburg as far as Irwin, and south along the Southwest Branch as far as Youngwood. A total of 50 000 people are supplied, in addition to large industrial and mining plants, by the companies in which he was the guiding spirit, and this, too, in a section where, because of the broken topography and the dumping of mine drainage into the streams, the operating conditions are exceedingly difficult and expensive. Mr. Forbes was in responsible charge of the design and construction of the plant, and, at the time of his death, he was, and had been for many years, Manager, Secretary, and Treasurer of the Company and its allied interests.

Mr. Forbes' success at Greensburg resulted in his being chosen to take charge of the construction and operation of water plants for the Derry Water Company at Derry, Pa., and the Dennison Water Supply Company at Dennison, Ohio, serving as Manager, Secretary, and Treasurer of both these corporations. He also had a large financial interest in all three companies.

Mr. Forbes was recognized as an authority in matters pertaining to water-works construction, operation, and valuation. He was employed in a number of water-works cases, either as an expert witness or as a member of boards of arbitration. He assisted in organizing the Pennsylvania Water Works Association and served for 3 years as its President. He was also a Member of the American Water Works Association, the New England Water Works Association, the Engineers' Society of Western Pennsylvania, the Engineers' Society of Pennsylvania, and the Union League Club of Philadelphia. He was a Mason and a member of the Protestant Episcopal Church.

Mr. Forbes was a delightful character, always kind and courteous, a man who made friends readily and who kept them. Of strong convictions, he was also a diplomat. Although for many years he had acted as Manager of the Water Company which served the town in which he lived—and under difficult conditions and with necessarily high rates—he was popular and highly respected in that community. An editorial in a Greensburg newspaper, at the time of his death, reads, in part, as follows:

"Mr. Forbes was in many ways superior. He was a factor in the affairs of his town and of his county. He was intensely human. He was a master of his profession, and he controlled because he knew. A useful man has been taken from this community."

In 1893, Mr. Forbes was married to Miss Ethel Parvin, of Philadelphia, who, with five children, survives him.

Mr. Forbes was elected an Associate Member of the American Society of Civil Engineers, on June 5th, 1907.

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"THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA." F. LAVIS. (To be presented April 1st, 1914.)

"THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS." J. A. L. WADDELL. (To be presented April 15th, 1914.)

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"Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors." JOHN R. NICHOLS ..... Apr., "
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"Flood Flows." WESTON E. FULLER ..... May, "
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PAPERS AND DISCUSSIONS

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THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE

By Charles H. Lee, Assoc. M. Am. Soc. C. E.

To be Presented May 6th, 1914.

Summary and Conclusions.

The objects of this paper are to show the possibility and practicability of measuring the annual rate of recharge of underground reservoirs of the closed-basin type, and to indicate broadly the factors which determine safe yield from a basin by artificial development, such as Artesian flow or pumping.

The paper opens by pointing out the importance of the problem in California and the Southwest. Following this there is a description of the physical features of underground reservoirs and the general principles of inflow, outflow, and storage. The body of the paper presents detailed methods and results of extended measurements, by the Los Angeles Aqueduct Bureau and the United States Geological Survey, for the determination of the rate of annual recharge of the Independence Basin, in Owens Valley, California. The subjects of percolation from stream channels, relation of precipitation and altitude, soil evaporation combined with transpiration from grass, and ground-water fluctu-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

ations, were carefully studied in the field, and original data are presented. The paper closes with a discussion of the relation which the net safe yield from a basin bears to the rate of annual recharge.

The conclusions are as follows:

1.—The "underground reservoirs" of California and the Southwest are water-tight rock basins, represented by the topographic valleys, which are filled with porous alluvial material in which the voids are saturated with water.

2.—Inflow into these basins is by percolation from water on the surface of the alluvial filling, which source may occur as direct precipitation, stream flow, irrigation, or flooding. Natural ground-water loss occurs in the region of lowest depression of a basin, and consists of the breaking out of water at the surface in springs or seepages, evaporation from soil, transpiration, and underflow. Artificial development, by wells or other methods, reduces the natural ground-water loss. Considered as averages, the rates of recharge and ground-water loss are equal, unless the artificial draft is excessive.

3.—The rate of recharge in a region of small precipitation and high evaporation rate can be determined most accurately, and with least expenditure of time and money, by measuring the elements which make up the ground-water loss. Of the natural elements, the most important are soil evaporation and transpiration. The underflow is relatively small and often negligible.

4.—The safe yield of artificially developed ground-water obtainable from an underground reservoir is less than indicated by the rate of recharge, the quantity depending on the extent to which soil evaporation and transpiration can be eliminated from the region of ground-water outlet.

Introduction.

There are in California and the arid States of the Southwest many valleys underlaid by porous alluvial material in which the voids are filled with water. The ease with which water can be developed from wells in these valleys and the definite bounds of the water-bearing formation have led to the use of such terms as "underground lake," "underground basin", or "underground reservoir". These terms are in general use among local hydraulic engineers, and have been adopted by the California Courts in numerous recent decisions pertaining to the use of diffused percolating water occurring in closed basins.

A problem which is being presented to the Engineering Profession for solution is the determination of the safe yield of "underground reservoirs", or the net annual supply which may be developed by pumping and Artesian flow without persistent lowering of the ground-water plane. The answer to this problem must soon be had throughout the Southwest, and particularly in Southern California, where the use of underground water has advanced most rapidly. The available surface supplies of the region are now used so extensively that future extension of irrigation must depend on the underground supply. Already, however, the growing popularity of ground-water supply for irrigation and the heavy drafts made possible by improved pumping machinery and cheap power are giving rise to conditions of dangerous overdraft on many of the so-called inexhaustible underground water supplies. Furthermore, in many of the sparsely settled valleys of the Southwest, where very limited ground-water supplies are available, preparations are being made to develop pumped water for irrigation far in excess of the safe yield. The writer has in mind such a valley, where, out of 90 000 acres of agricultural land, filed on in good faith under the provisions of the Desert Land and Homestead Acts, it can be said with reasonable certainty that not more than 2% can ever be put under cultivation. In addition to the use of underground water for irrigation, it is being developed extensively for municipal purposes. The City of Los Angeles derives its present supply entirely from an underground reservoir, the San Fernando Valley, and is preparing to develop a similar supply in Owens Valley to be held as a reserve in connection with the Los Angeles Aqueduct. A portion of the supply of both Oakland and San Francisco is developed from underground sources, and the possibility of increasing largely the ground-water supply derived from Livermore Valley for the latter city has been the subject of considerable debate among prominent members of the Society. The problem, therefore, is an important one, and on it depends, not only the safe investment of capital, but also the very life of large industries and communities.

The sources of underground water are so difficult of measurement and its movements are so hidden from view, that the solution of the problem, until very recently, has been merely a subject for speculation and theory. Within the past few years, however, the study of underground water supply has been given considerable attention by the

United States Geological Survey as well as by engineers who have had these problems to meet. Although the fact of the existence of these "underground reservoirs" has been established, their sources of supply and outlets recognized, and many data regarding well fluctuations have been accumulated, yet very little has been done toward developing methods of measuring the rate of recharge or studying the factors which limit the quantity of water which can be safely developed from underground reservoirs.

The writer has had opportunity to investigate a number of the important underground reservoirs of California, and in this paper he presents certain general principles which seem to him to be justified by the existing data. Although these principles may seem to be self-evident, yet the writer has no knowledge that they have ever been applied to the practical solution of the problem in hand. To show the possibilities of their application, therefore, he presents data and studies for an underground reservoir in Owens Valley, California, where it was desired to ascertain the quantity of ground-water that could be developed safely without overdraft. Much of this information has already appeared in print* in greater detail, but the writer believes that the subject is of sufficient importance, and the component studies are of wide enough technical interest to be presented to the members of the Society for discussion and expression of opinion.

General Principles.

The typical underground reservoir is, geologically, a structural basin filled with alluvial débris from the adjoining mountain ranges. These basins are the product of faulting accompanied by the uptilting of a crustal block from one side of the line of fracture. The formation is very common throughout the Southwest, reaching its most perfect development in the Great Basin region of Utah and Nevada, where the name "Basin-Range" has been applied to it. In California the basins are found in the valleys of the Coast Range and along the base of the Sierra Nevada, Sierra Madre, San Bernardino, and San Jacinto Ranges. The rock enclosing these basins is in most cases impervious to water and practically insoluble. Along the coast of California, shales and cemented gravels predominate, and are practically non-water-bearing in comparison with the porous gravels

* Water Supply Paper No. 294, U. S. Geological Survey, 1912.

filling the basins; and, in the interior of the State, the enclosing rock formation is largely granite. Most of the basins can be considered as closed except for a subterranean outlet usually known as the "Narrows". This occurs at the lowest point in the rock rim, where the gravels contract into a neck filling a narrow depression or canyon cut into the confining rock. The quantity of underground water escaping through such an outlet is usually very small, however, as has been shown by a number of well-known underflow observations. Hence, the underground reservoirs can generally be considered as closed rock basins, the effective storage capacity of which is the void spaces between the particles of sand and gravel with which they are filled.

The usual sources of supply for underground reservoirs are percolation from flowing surface streams, from precipitation, or, where the supply is not ground-water derived from the basin, from irrigation on the surface of the porous gravels. The water thus absorbed sinks downward to the general ground-water plane and then moves laterally toward the region of lowest depression. This region, in contrast to the surrounding dry soil or desert, is usually characterized by springy, swampy conditions, and is commonly known in Southern California as a cienaga. The natural outlets for underground water are by springs or seepages discharging into the surface channels which drain the cienaga, by evaporation from damp soils and vegetation within the cienaga, and, to a limited extent, by underflow from the basin. The surface streams formed by the oozing out of underground water join to form a larger stream, which in all respects corresponds to the outlet of a lake or reservoir, and, passing from the basin, pursues its course just as any other surface stream. Its flow is characterized by permanence and regularity, except as it is augmented by surplus flood water which, during a limited period following winter storms, passes from the basin without being absorbed by the gravels.

The general principles of inflow into and outflow from an underground reservoir of the type described correspond with those of surface reservoirs. The difference lies in the relative speeds with which the general water surface assumes a horizontal position following increase or decrease of volume stored. In the case of a surface reservoir or lake, the effect of inflow or outflow is an immediate complete readjustment of surface level. The frictional resistance offered by the

particles filling an underground reservoir is so great, however, that the movement of water from an area of high level is very slow, varying from a few hundred feet to a few feet per day, depending on local conditions. As a result, the water surface in an underground reservoir is never horizontal, being steepest near the mouths of the mountain canyons, the run-off from which is the most important source of supply; it is most nearly horizontal at the region of outlet; and varies in slope and elevation from time to time, depending on the rate of recharge.

The average rates of inflow and outflow of an underground reservoir must be equal, otherwise there would be persistent rise or fall of ground-water levels until such a balance is reached. There are, therefore, two possible methods of measuring the rate of recharge, either by determining the total percolation from various sources into the porous material of the basin, or by determining the ground-water losses. The first method is to be preferred where the source of percolation is almost entirely stream flow from which channel losses can be accurately measured; or where the precipitation is large, well distributed through the year, and forms the principal source of supply. The first of these conditions could occur only in an arid region, and the second is typical of humid regions.

The method by determination of ground-water losses is one peculiarly adapted to arid or semi-arid conditions with high evaporation rate, such as exist throughout the Southwest. It has been the writer's observation in this region that soil evaporation and transpiration constitute from 50 to 100% of the ground-water losses from underground reservoirs, the average exceeding 75 per cent. Other losses are largely the flow from springs and seepages, which can be measured with precision. Rates of soil evaporation and transpiration from grasses do not present insurmountable difficulties of measurement under arid conditions. In fact, it has been the writer's experience that satisfactory results with specially designed equipment could be obtained from observations extending over 2 years, although a period of 3 years is preferable. Furthermore, the area from which evaporation occurs and the depth to ground-water at various points within it are not subject to wide fluctuations, and are easily measured. The determination of the rate of recharge of underground reservoirs of the basin type, therefore, is a problem of soil evaporation, transpiration, and stream

flow, all of which processes, with the exception of transpiration from trees, are now capable of measurement with relative accuracy at reasonable cost.

The general method pursued in the Owens Valley studies was to ascertain, by extended field measurements of soil evaporation, transpiration, and spring discharge, the average rate of outflow from the basin. All available evidence seemed to indicate that the basin was closed, so that the rate of outflow equalled the rate of inflow or recharge. As a check, therefore, the rate of inflow into this basin from precipitation, stream flow, and irrigation was also determined. The data are presented under the following headings: Physical Features, Precipitation, Stream Flow, Evaporation and Transpiration, Groundwater, and Rate of Recharge by Percolation.

Physical Features.

General.—The Owens Valley lies in east-central California, along the western border of the Great Basin, and at the base of the steep slope of the Sierra Nevada Mountains, as shown by Fig. 1. Including a northern extension, known as Long Valley, its length is 120 miles, and its width, from crest to crest of confining mountain ranges, varies from 15 to 40 miles. The total area of the valley and its tributary mountain drainage is about 3 300 sq. miles, of which 1 200 sq. miles are desert mountains from which the run-off is negligible, 536 sq. miles comprise the Sierra Nevada slope, which yields a large run-off, and 1 580 sq. miles are the transition slope and valley floor, from which very slight surface run-off occurs. The elevation of the valley floor varies from 8 000 to 3 570 ft. above sea level, the latter being at Owens Lake, the lowest depression of the valley. The average elevation of the crest of the Sierra Nevada is 12 500 ft., with many peaks exceeding this elevation by more than 1 500 ft. The White and Inyo Mountains, a desert range bordering the valley on the east, have an average elevation of 10 000 ft., with peaks reaching 13 000 ft.

The valley is a deep structural trough filled with porous alluvial material derived principally from the Sierra Nevada, and inclosed by impervious rock formations. The steep east face of the Sierra Nevada is the result of faulting accompanied by elevation and westward tilting of a great crusted block. The drainage system of the valley consists of a trunk stream, Owens River, fed by approximately

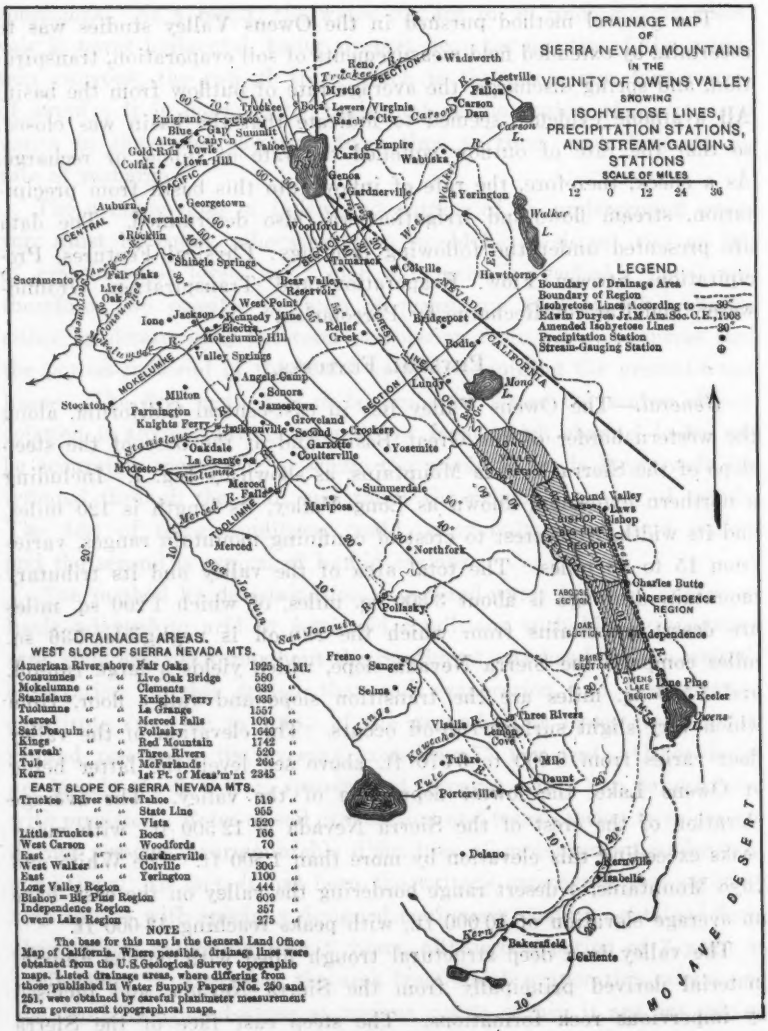

DRAINAGE MAP
OF
SIERRA NEVADA MOUNTAINS
IN THE
VICINITY OF OWENS VALLEY
SHOWING
ISOHYETOSE LINES,
PRECIPITATION STATIONS,
AND STREAM-GAUGING
STATIONS
SCALE OF MILES
12 0 12 24 36
LEGEND
Boundary of Drainage Area
Boundary of Region
Isohyetose Lines According to Edwin Duryea Jr. M. Am. Soc. C. E., 1908
Amended Isohyetose Lines
Precipitation Station
Stream-Gauging Station
DRAINAGE AREAS.
WEST SLOPE OF SIERRA NEVADA MTS.
American River above Fair Oaks 1925 Sq. Mi.
Consumnes " " Live Oak Bridge 560 "
Mokelumne " " Clements 639 "
Stanislaus " " Knights Ferry 935 "
Tuolumne " " La Grange 1557 "
Merced " " Merced Falls 1090 "
San Joaquin " " Pollasky 1040 "
Kings " " Red Mountain 1742 "
Kaweah " " Three Rivers 520 "
Tule " " McFarlands 266 "
Kern " " 1st Pt. of Mea'm'nt 2345 "
EAST SLOPE OF SIERRA NEVADA MTS.
Truckee River above Tahoe 519 "
" " " State Line 955 "
" " " Vista 1520 "
Little Truckee " " Boca 166 "
West Carson " " Woodfords 70 "
East " " " Gardnerville —— "
West Walker " " Colville 306 "
East " " " Yerington 1100 "
Long Valley Region 444 "
Bishop = Big Pine Region 609 "
Independence Region 357 "
Owens Lake Region 275 "
NOTE
The base for this map is the General Land Office Map of California. Where possible, drainage lines were obtained from the U.S. Geological Survey topographic maps. Listed drainage areas, where differing from those published in Water Supply Papers Nos. 250 and 251, were obtained by careful planimeter measurement from government topographical maps.
Reno
Wadsworth
Truckee
Mystic
Lovers
Virginia City
Fallon
Carson Dam
Carson L.
Empire
Wabuska
Yerington
Walker L.
Genoa
Gardnerville
Woodford
Tamarack
Colville
Hawthorne
Bear Valley Reservoir
West Point
Kennedy Mine
Electra
Mokelumne Hill
Relief Creek
Bridgeport
Bodie
Lundy
Mono L.
Valley Springs
Angels Camp
Sonora
Jamestown
Milton
Stockton
Farmington
Knights Ferry
Jacksonville
Second
Crockers
Groveland
Garrotte
Coulterville
Yosemite
Oakdale
La Grange
Modesto
Merced
Merced Falls
Summerdale
Mariposa
Northfork
Pollasky
Fresno
Sanger
Selma
Visalia
Three Rivers
Lemon Cove
Tulare
Milo
Daunt
Porterville
Delano
Kernville
Isabella
Bakersfield
Caliente
Round Valley
Laws
Bishop
Big Pine
Keeler
Owens
Charles Butte
Independence
MOJAVE DESERT
Auburn
Georgetown
Newcastle
Rocklin
Shingle Springs
Fair Oaks
Live Oak
Sacramento
Ione
Jackson
Electra
Mokelumne Hill
Valley Springs
Angels Camp
Sonora
Jamestown
Milton
Stockton
Farmington
Knights Ferry
Jacksonville
Second
Crockers
Groveland
Garrotte
Coulterville
Yosemite
Oakdale
La Grange
Modesto
Merced
Merced Falls
Summerdale
Mariposa
Northfork
Pollasky
Fresno
Sanger
Selma
Visalia
Three Rivers
Lemon Cove
Tulare
Milo
Daunt
Porterville
Delano
Kernville
Isabella
Bakersfield
Caliente
Round Valley
Laws
Bishop
Big Pine
Keeler
Owens
Charles Butte
Independence
MOJAVE DESERT

FIG. 1.

forty tributaries entering at fairly regular intervals from the west. The river terminates in Owens Lake, an alkaline body of water without outlet which disposes of all surplus surface water from the valley by evaporation. Irrigation is necessary for the production of crops, and water is diverted in canal systems from Owens River and tributary streams.

Independence Basin.—The portion of Owens Valley which is the subject of this study will be spoken of as the Independence Basin (Plate II). It lies in the south-central part of the valley, embracing with its tributary drainage area the region between the Sierra Nevada crest and Owens River, lying between Poverty Hills on the north and Alabama Hills on the south. These latter are secondary ranges extending into the valley from the Sierra Nevada and isolating a portion of the valley fill about 25 miles long and from 6 to 8 miles wide.

For the purpose of this study, the surface of the region has been classified as high mountain drainage, intermediate mountain slopes, outwash slope, and valley floor. (Table 1 and Plate II.)

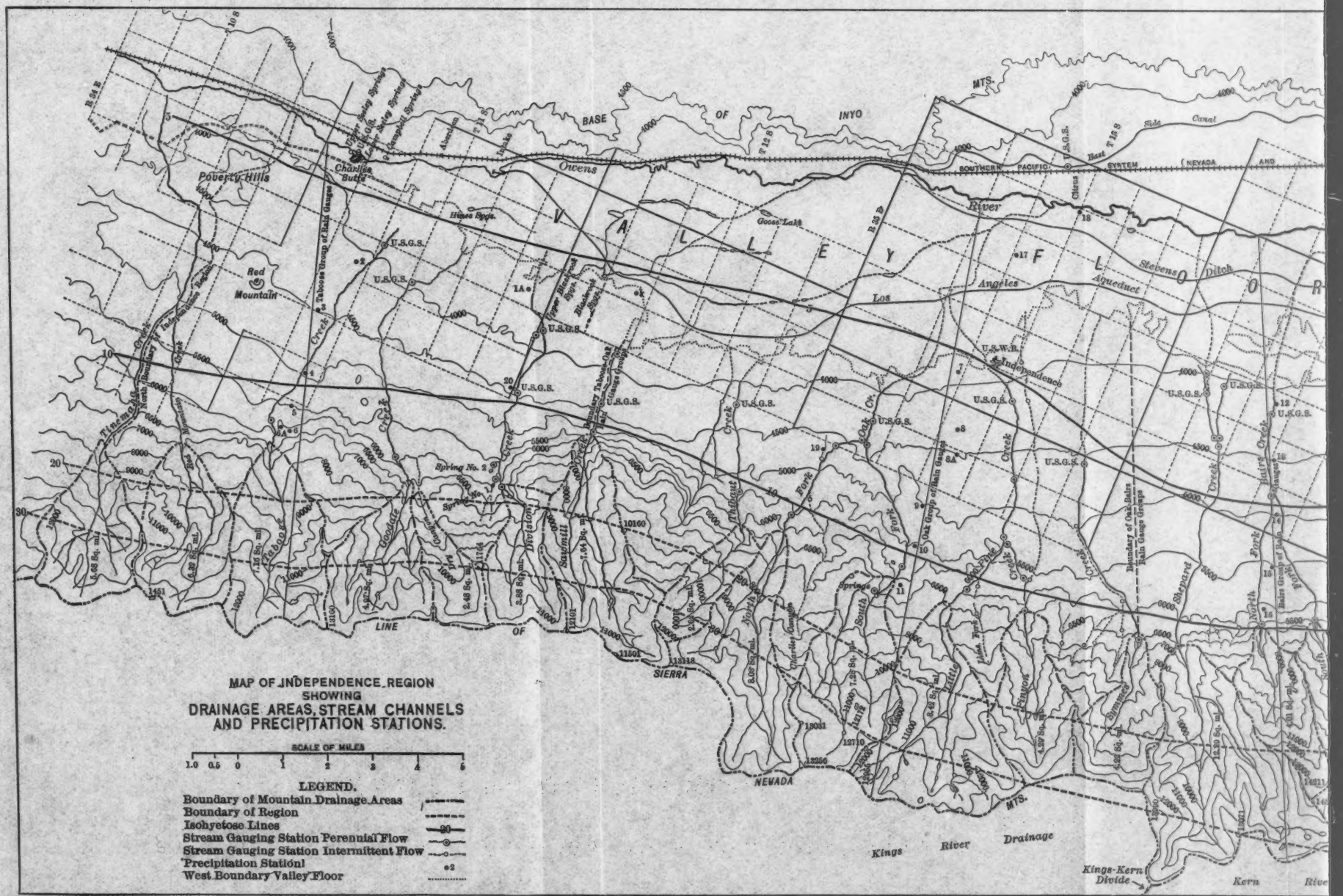
TABLE 1.—SUBDIVISIONS OF INDEPENDENCE REGION.

Subdivision.	Area.		Boundary.		Slope.	Vegetation.	Character of surface.
	Square miles.	Percentage of total.	Upper.	Lower.			
High mountain drainage.	96.0	27	Sierra crest.	Mouth of canyon.	Precipitous to gentle.	Isolated forest trees.	Bare granite and fragmental rock accumulations.
Intermediate mountain slope.	29.4	8	Canyon drainage.	6 500-ft. contour.	2000 to 3000 ft. to the mile.	Desert bushes and nut pine.	Fragmental and finely disintegrated rock accumulations.
Outwash slope.	165.3	46	6 500-ft. contour.	Grass land.	300 to 600 ft. to the mile.	Desert bushes.	Boulders, sand and gravel.
Valley floor:							
Cultivated....	4.7	1	.....	.....	Gentle.....	Alfalfa, etc....	Soil.
Meadow.....	45.1	13	.....	.....	Gentle to level.	Salt grass, etc.	Soil.
Alkali.....	2.7	1	.....	.....	Level.....	None.....	Soil.
Desert..	13.9	4	.....	.....	Level.....	Desert bushes.	Fine sand.
	357.1	100					

The high mountain drainage embraces the eastern slope of the Sierra Nevada and consists of a series of seventeen small canyons which are the drainage basins of streams tributary to Owens River (Table 2). These canyons are all narrow at the mouth (the 6 500-ft. level) and broaden out more or less toward the summit, presenting a roughly triangular shape. They are separated by high knife-edge ridges, which terminate in triangular slopes facing the valley. They have been cut by water erosion and sculptured by active glaciation above the 7 500-ft. level, their upper portions being well-developed glacial cirques. In many places below the cirques are series of benches occupied by glacial lakes or meadows. Most of the cirque floors are buried beneath morainal accumulations; some of the polished canyon bottoms between the 11 500 and 8 000-ft. levels are swept clean of débris, and others are completely buried by morainal material. Terminal and lateral moraines of considerable size occupy the canyon floors between the 8 000 and 7 000-ft. levels.

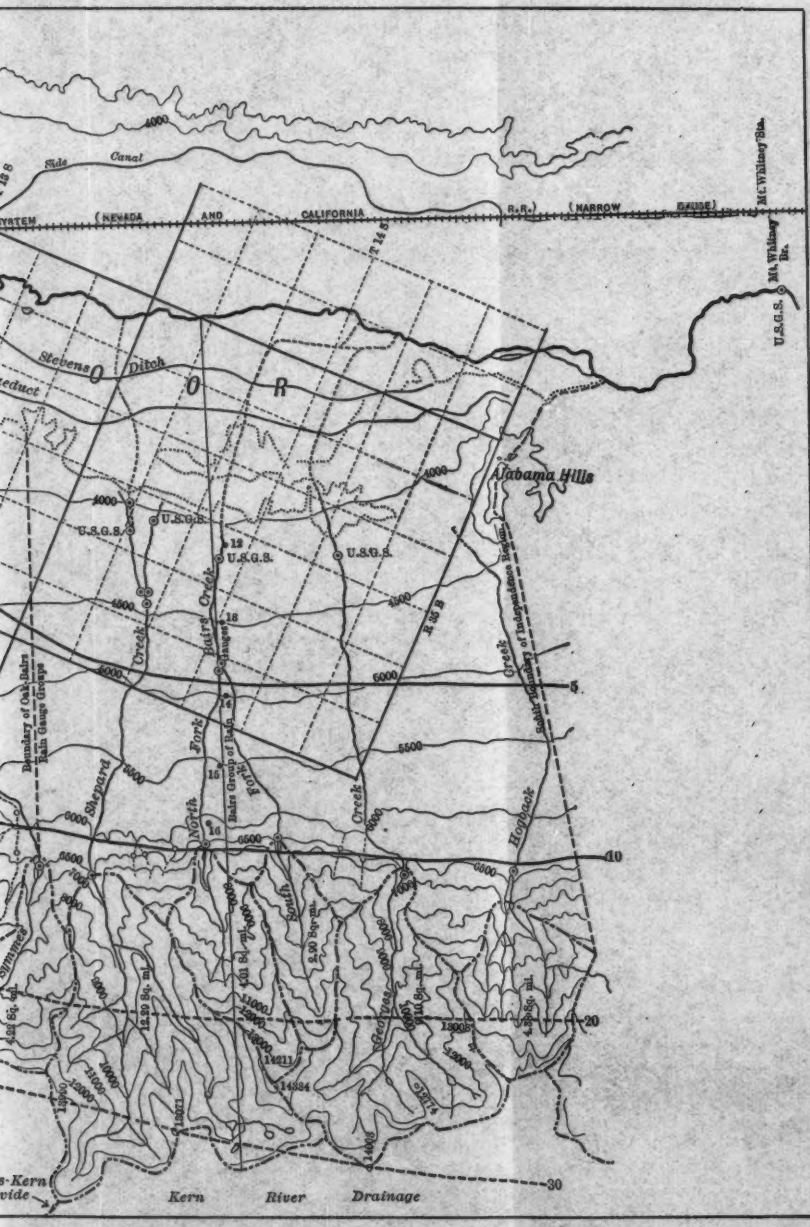
TABLE 2.—HIGH MOUNTAIN DRAINAGE AREAS OF INDEPENDENCE REGION.

Creek.	Area.		Elevation.		Shape.	Length of Sierra crest drained, in miles.	Remarks.
	Total, in square miles.	Percentage above 10 000 ft.	Head of canyon, in feet.	Mouth of canyon, in feet.			
Taboose	7.16	60	12 000	6 500	Triangular...	3.34	Morainal deposits; regulated run-off.
Goodale	4.97	69	12 500	6 500	Triangular...	2.67	Morainal deposits; regulated run-off.
Dry Canyon	2.48	65	12 000	6 500	Triangular...	1.21	Morainal deposits; no surface run-off.
Division	3.88	51	12 000	6 000	Rectangular.	1.88	Morainal deposits.
Sawmill	7.64	44	12 000	5 000	Rectangular.	2.75	
Thibaut (N. Fk.).	2.25	20	11 500	6 000	Irregular....	0.0	
Thibaut (S. Fk.)..	2.62	55	12 000	6 000	Irregular....	0.0	
Oak (N. Fk.)	8.08	65	12 500	6 000	Irregular....	5.17	Morainal deposits; regulated run-off.
Oak (S. Fk)	7.28	57	12 500	6 000	Irregular....	1.06	
Little Pine	8.42	74	13 000	6 500	Triangular...	4.60	
Pinyon	4.29	47	13 000	6 500	Irregular....	1.09	
Symmes	4.22	43	13 000	6 300	Triangular...	1.59	
Shepard	12.29	66	13 500	6 500	Triangular...	7.95	5.68 miles of crest south of Kings-Kern divide.
Bairs (N. Fk.)	4.01	43	13 500	6 300	Irregular....	0.0	Lies on east face of Mount Williamson.
Bairs (S. Fk.)	2.90	41	13 000	6 300	Irregular....	0.0	Lies on east face of Mount Williamson.
George	9.10	74	13 500	6 500	Triangular...	3.89	
Hogback	4.38	58	13 000	7 000	Irregular....	0.67	
	95.97	55	.....	.....	.....	37.87	


MAP OF INDEPENDENCE REGION
SHOWING
DRAINAGE AREAS, STREAM CHANNELS
AND PRECIPITATION STATIONS.
SCALE OF MILES
1.0 0.5 0 1 2 3 4 5
LEGEND.
Boundary of Mountain Drainage Areas
Boundary of Region
Isohyetose Lines
Stream Gauging Station Perennial Flow
Stream Gauging Station Intermittent Flow
Precipitation Station
West Boundary Valley Floor
VALLEY
BASE OF INYO MTS.
SIERRA NEVADA MTS.
LINE OF
Owens River
Kings River Drainage
Kings-Kern Divide
Kern River
Los Angeles Aqueduct
Stevens Ditch
SOUTHERN PACIFIC SYSTEM (NEVADA AND
Poverty Hills
Red Mountain
Charles Butte
Haiwee Res.
Goose Lake
Independence
Oak Creek
Fish Creek
Shepard Creek
Baker Creek
Tinemaha Creek
Goodale Creek
Division Creek
Symmes Creek
Hunt Creek
Pinyon Creek
Fairview Valley
Spring No. 2
U.S.G.S.
U.S.W.B.

BASE OF INYO MTS.
Owens
VALLEY
River
Goose Lake
Los Angeles Aqueduct
SOUTHERN PACIFIC SYSTEM
Olancha
Independence
Oak Creek
Oak Flat
Oak Grove Pack Stock Station
Boundary of Oak Basin Rain Gauge Groups
Upper Blackrock Spring
Blackrock Spring
Rain Gauge Groups
Shepherd Creek
Rainbow Tahoma Oak Flat
Rain Gauge Groups
Sawmill
Thibaut
Fork
North
South
Fork
Spring
Creek
Charlies Gulch
Lone Pine
Creek
Lone Pine
Creek
Lone Pine
Creek
Lamb
Canyon
Tuttle
Creek
Symmes
Creek
SIERRA
NEVADA
MTS.
Kings River Drainage
Kings-Kern Divide

PLATE II.
PAPERS, AM. SOC. C. E.
APRIL, 1914.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.



The intermediate mountain slopes (Fig. 2) are the triangular areas terminating the ridges between the canyons, and probably represent the original face of the range before it had been actively eroded (Table 3). Their lower boundary has been arbitrarily placed at the 6 500-ft. contour, and their apexes reach a maximum elevation of about 12 000 ft. They have a steep uniform slope of from 2 000 to 3 000 ft. to the mile, and in general are covered with a mantle of disintegrated rock and slide material which merges into the valley fill.

TABLE 3.—INTERMEDIATE MOUNTAIN SLOPES OF INDEPENDENCE REGION.

Adjoining high mountain drainage areas.	Area, in square miles.	Elevation.			Distance from Sierra crest to center of area, in miles.	Remarks.
		Apex, in feet.	Center of area, in feet.	Lower border, in feet.		
Tinemaha	2.17	(11 000)	8 000	6 500	3.0	
Red Mountain	2.37	(12 000)	8 300	6 500	3.0	
Taboose	3.94	12 200	8 000	6 500	3.3	
Goodale	2.29	11 800	7 200	6 500	2.6	Does not include Dry Canyon.
Division	0.95	9 500	7 500	6 500	3.5	
Sawmill	1.32	10 200	7 500	6 500	3.2	
Thibaut (North Fork)	0.53	10 500	7 500	6 500	3.1	
Thibaut (South Fork)	0.07	7 000	6 700	6 500	4.0	Charlies Canyon yields run-off in normal and above normal years.
Oak (North Fork)	3.62	12 600	7 100	6 500	4.2	
Oak (South Fork)	1.03	10 600	7 100	6 500	3.8	
Little Pine	2.02	11 800	7 400	6 500	3.8	Lime Fork yields run-off in normal and above normal years.
Pinyon	2.89	11 500	7 600	6 500	2.7	
Symmes	0.42	9 200	7 400	6 500	3.2	North Fork similar to Lime Fork.
Shepard	0.97	9 900	7 200	6 500	4.5	
Bairs (North Fork)	0.48	9 100	7 100	6 500	5.0	
Bairs (South Fork)	1.21	10 300	7 800	6 500	4.6	
George	2.09	11 200	8 100	6 500	3.5	
Hogback (one-half)	1.08	10 800	7 900	6 500	4.1	
	29.45	.....	.....	.....	.....	

The outwash slope (Fig. 2) is the desert portion of the surface of the valley fill, extending from the 6 500-ft. contour at the base of the Sierra Nevada to the upper edge of grass and irrigated land in the valley (3 900 to 4 000 ft.). Its surface is composed of loose boulders, gravel, and sand, deposited during past ages by torrential streams coming from the mountains. This deposit is of

unknown depth, and lies on a buried ancient rocky surface, the higher hills of which appear above the present surface as buttes or knolls. The channels of streams draining the mountain canyons cross this slope in trenches, which, near the mountains, are from 25 to 50 ft. deep.

The valley floor embraces the area between the outwash slope and Owens River, and its surface may be classified as irrigated land, grass or meadow land, alkali land, and desert. The upper edge has a maximum slope of about 120 ft. to the mile, but within a short distance it merges into the practically level valley. The surface is soil to a depth of from 1 to 3 ft., except on the desert land, where it is fine sand. Most of the irrigated land is along the upper margin of the valley floor adjoining the creek channels. The grass or meadow lands lie between and to the east of the ranches, and extend well out into the level valley. The growth is most luxuriant in the spring zone, which is about ¼ mile wide and is at the upper edge of the valley floor. Here are numerous small flowing springs, with temperature of about 62°, which start the meadow grass early in the season and keep it green until late in the autumn. Farther out in the valley, the salt grass makes a green carpet from May until late July. In the salt-grass land there is always a deposit of alkali about the plant roots, and the soil surface is crusted. The spring zone, however, is free from alkali. The worst alkali land is practically bare of vegetation and is thickly crusted with white salts. It lies in the more level areas in the center of the valley.

The desert area to the east of Owens River yields no appreciable run-off, and, owing to its light precipitation, it makes no contribution to the ground-water.

The alluvial material which forms the valley fill varies in size from large boulders to fine clay, and, in arrangement, from a thorough mixture of all sizes to layers of well-assorted gravel, sand, and clay. The transporting medium was water, both mountain streams and Owens River taking part in the work. Some of the material was deposited in the beds and on the sides of shifting stream channels, and much of the finer sand and clay was deposited from the quiet waters of a large lake which occupied the lower portion of the valley. The structure of the valley fill, therefore, is complex, and the character of the alluvial material underlying a given locality is difficult to determine without actual examination from borings.

FIG. 3.—EVAPORATION FAN IN OWENS RIVER.

FIG. 4.—EVAPORATION FAN IN SOIL.

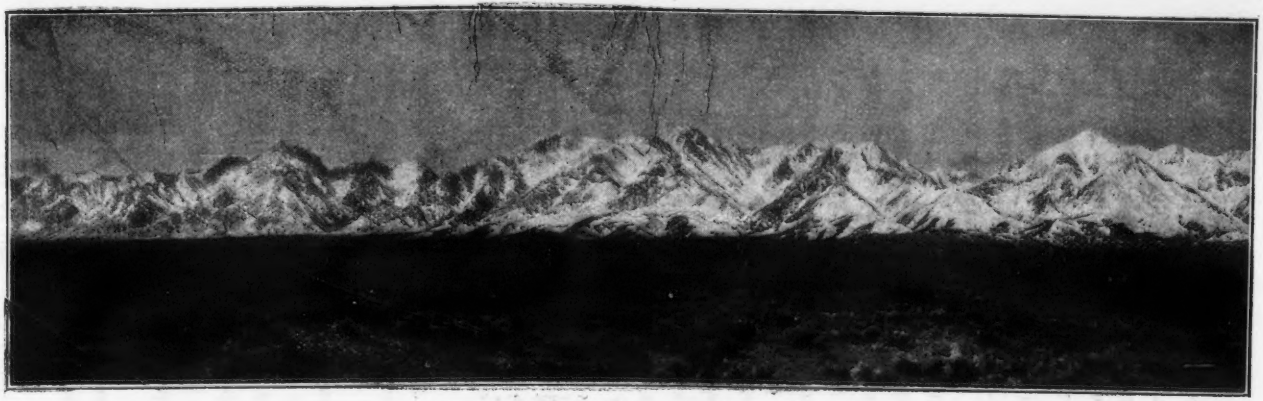


FIG. 2.—EASTERN SLOPE, SIERRA NEVADA.

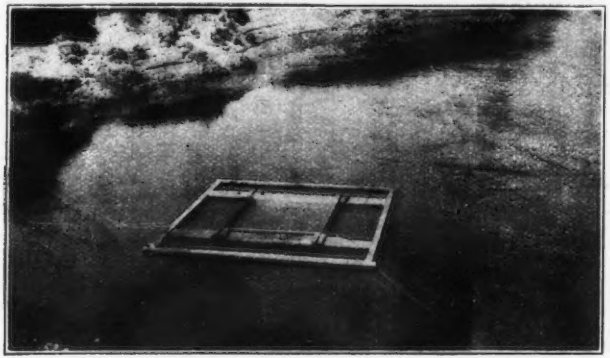


FIG. 3.—EVAPORATION PAN IN OWENS RIVER.

FIG. 4.—EVAPORATION PAN IN SOIL.

A number of borings, ranging in depth from 250 to 500 ft., have been made in the basin by the City of Los Angeles, in connection with the development of the aqueduct supply. In general, the materials encountered were clay, sand, and coarse gravel in layers varying in thickness from a few inches to 150 ft. Coarse material in thin layers interbedded with clay predominates along the upper edge of the valley floor in the spring belt. All wells in this belt yield Artesian flows of from 1 to 2 sec-ft. The material is progressively finer and occurs in thicker strata east of this belt, toward the center of the valley, and the Artesian flows decrease in volume. Near Owens River, fine sand and clay in alternate layers is the only material encountered above the 300-ft. depth.

The streams from the Sierra Nevada were by far the most active in the work of building up the valley fill. Their loads were acquired in the mountain canyons and carried out into the valley, where they were dropped in order of size as the velocity of flow decreased. The old lake level stood at an elevation of about 3 790 ft. for a long period, as shown by beach lines on the east slope of the Alabama Hills. The present 3 790-ft. contour lies near the spring and Artesian belt. The finer materials between the spring belt and Owens River are evidently lake deposits. The ancient lake was contemporaneous with other geologic lakes of the Great Basin, such as Lakes Bonneville and Lahontan. The geologic history of these lakes shows many wide fluctuations of water level, covering long periods of time. The interbedding of fine and coarse material encountered in the spring belt is evidently the result of such fluctuation, as the sudden checking of the velocity of a stream on entering a body of still water results in the immediate deposition of coarse material. The Artesian and spring conditions, therefore, result from hydrostatic pressure on the water entrapped in these wedges of coarse material.

Two cross-sections of the valley, showing the probable geologic structure, were constructed along the Thibaut and Independence Sections (Fig. 5). The topography for these sections was obtained from the U. S. Geological Survey's map of the Mount Whitney quadrangle, and the character of the surface material was determined by field inspection. The exposed slopes of bed-rock on each side of the valley were joined beneath the valley floor, and the arrangement of the material filling the basin thus formed was represented according to

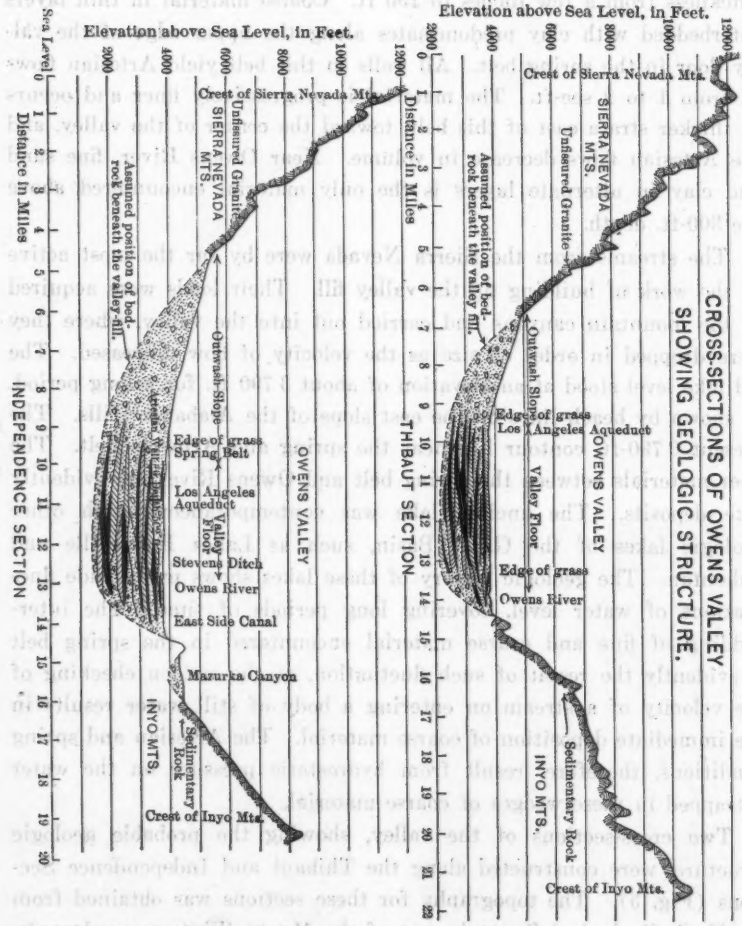

CROSS-SECTIONS OF OWENS VALLEY
SHOWING GEOLOGIC STRUCTURE.
Elevation above Sea Level, in Feet.
Crest of Sierra Nevada Mts.
SIERRA NEVADA MTS.
Unfissured Granite
Assumed position of bed-rock beneath the valley fill.
Outwash Slope
Edge of grass
Los Angeles Aqueduct
OWENS VALLEY
Valley Floor
Edge of grass
Owens River
Sedimentary Rock
INYO MTS.
Crest of Inyo Mts.
Distance in Miles
THIBAUT SECTION
Sea Level
Elevation above Sea Level, in Feet.
Crest of Sierra Nevada Mts.
SIERRA NEVADA MTS.
Unfissured Granite
Assumed position of bed-rock beneath the valley fill.
Outwash Slope
Edge of grass
Spring Belt
Los Angeles Aqueduct
Valley Floor
Stevens Ditch
Owens River
East Side Canal
Mazurka Canyon
Sedimentary Rock
INYO MTS.
Crest of Inyo Mts.
OWENS VALLEY
Distance in Miles
INDEPENDENCE SECTION

FIG. 5.

the best available knowledge, the strata of fine material being indicated by solid black. On the diagram, the greatest depth of alluvial filling measures 2 500 ft. in the Independence Section, and 1 800 ft. in the Thibaut Section. Two of the aqueduct wells near the Independence Section reached depths of 500 ft. in alluvial material, and a well drilled by the Southern Pacific Company, opposite the Alabama Hills at Lone Pine Station has reached a depth of 832 ft., entirely in fine sand. There is no reason to suppose that the gravel filling near Independence is less than 2 000 ft. in depth.

The volume of void space, or porosity, of a body of alluvial material of this type is variously estimated by different authorities at from 20 to 35% of the total. Samples of the mixed gravel, sand, and silt of the outwash slopes west of Independence were removed to a depth of 4 ft., without disturbing the natural arrangement of the particles, and weighed dry and after saturation. The results of these tests indicate a porosity of 28% for these samples. The presence of very coarse gravel and boulders in this material would reduce the porosity, and, for the valley fill as a whole, 25% is probably more nearly correct.

Precipitation.

The plan followed in the study of precipitation was to gather and assemble data from which to prepare isohyets for the basin. These appear on Plate II, and are based on the available precipitation data and an intimate knowledge of the local topography and vegetation.

Observations of precipitation were made in the Independence Basin as early as 1865, under the direction of United States Army officers stationed at Fort Independence. The record extends unbroken from September, 1866, to August, 1877, and was obtained under conditions sufficiently similar to permit of combining it with the more recent Weather Bureau record at Independence. The latter covers the periods from September, 1892, to August, 1895, and from September, 1898, to August, 1910, so that there are 26 seasons for which precipitation records are available at Independence. To supplement this record, twenty standard Weather Bureau rain gauges were established, and observations were made during the seasons 1908-09 and 1909-10 (Plate II). These gauges were distributed systematically over the valley floor and outwash slopes, and could all be reached during one day by three mounted observers stationed at points

in the valley where shelter was available. Four records were also available in Owens Valley, outside of the Independence Basin, at Bishop, Lone Pine, Laws, and Keeler. The Bishop and Lone Pine records are kept by co-operative Weather Bureau observers, and cover 15 and 5 years, respectively. The Laws and Keeler records are kept by railroad agents, and are for 13 and 24 years.

The distribution of total precipitation, with respect to geographic location, in the Independence Basin and adjoining areas depends to a great extent on topographic features, notably mountain ranges and valleys, although a consistent variation is also evident with changes in latitude. The controlling topographic feature is the Sierra Nevada, which has a general northwest and southeast trend.

This relation of precipitation and topography is well shown by studying observations made along cross-sections of the Sierra Nevada laid out at right angles to the trend of the range. Two such sections are indicated on Fig. 1 as the Central Pacific and Mokelumne Sections. The relations of mean annual precipitation, altitude, topographic position, and profiles of ground surface are presented graphically for the two sections in Diagrams 1 to 6 of Plate III. The marked similarity in the curves for the two sections indicates that the quantity of precipitation at points in a transverse section of the range conforms to some general law. Elevation, obviously, is not the controlling factor, for above the 5 000-ft. level the precipitation decreases with increase in altitude. The slope of the ground surface appears to be the most important element involved, as is seen from Diagrams 2, 3, 5, and 6 of Plate III. The phenomenon results from the condensation of aqueous vapor due to adiabatic cooling of masses of moist air driven up the slope of a mountain range by the prevailing winds. The region of maximum precipitation is at the lower cloud limit on the windward slope of the range, and above this the latent heat liberated by condensation raises the temperature above the dew point, resulting in decreased precipitation. After crossing the summit of a high range, the descending mass of air contracts in volume, thereby raising the temperature rapidly above the dew point and resulting in marked decrease of precipitation.

The increase of precipitation with elevation was first observed by Mr. S. A. Hill.* in studying rainfall in the northwest Himalayas of

* "California Hydrography," by J. B. Lippincott, M. Am. Soc. C. E., Water Supply Paper No. 81, U. S. Geological Survey, p. 354.

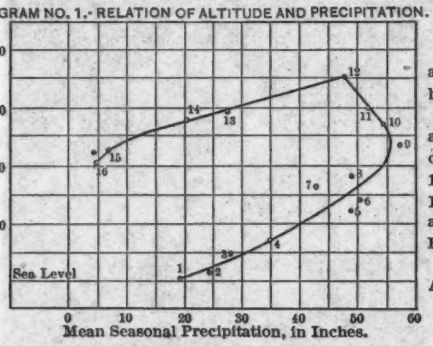
DIAGRAMS SHOWING THE

INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON P

AS ILLUSTRATED BY THE SIERRA NEVADA MTS.

CENTRAL PACIFIC GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 1.- RELATION OF ALTITUDE AND PRECIPITATION.



NOTES

Observations on Central Pacific and Mokelumne Groups of gauges by U.S. Weather Bureau.

Observations on Taboose, Oak, and Bairs Groups of gauges by City of Los Angeles. (See Tables 16 and 17.) For location of gauges see Drainage Map of Sierra Nevada Mts. and Drainage Map of Independence Region.

The Season is from Sept. 1st.to Aug. 31st.

DIAGRAM NO. 2.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

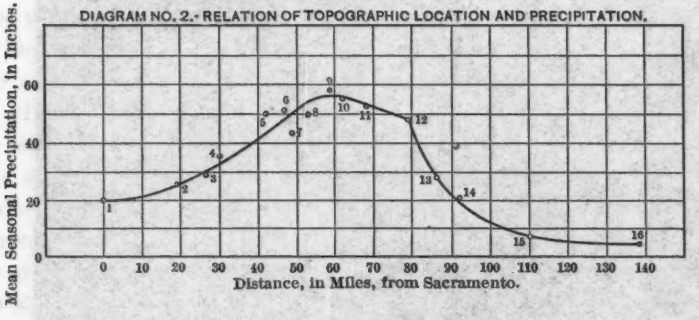
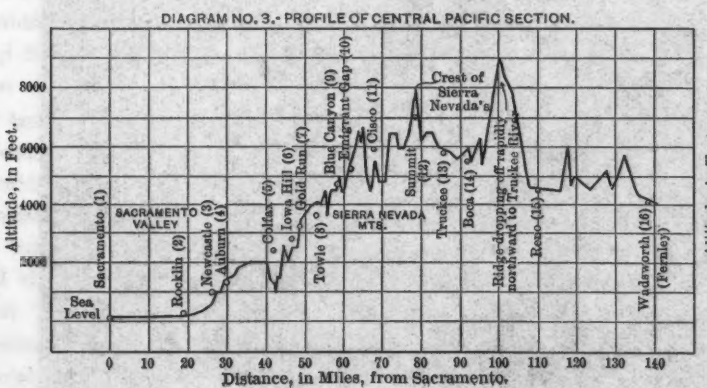


DIAGRAM NO. 3.- PROFILE OF CENTRAL PACIFIC SECTION.



MOKELUMNE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 4.- RELATION OF ALTITUDE AND PRECIPITATION.

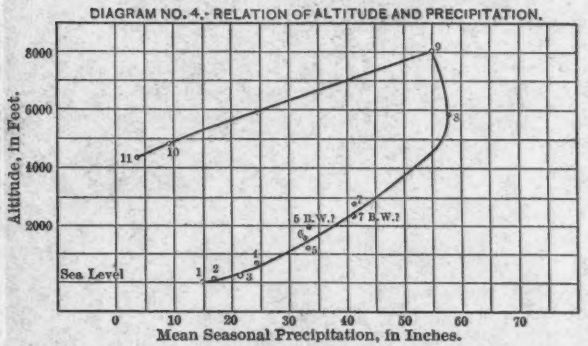


DIAGRAM NO. 5.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

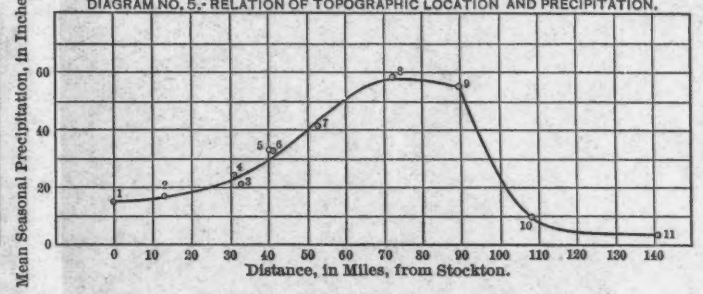
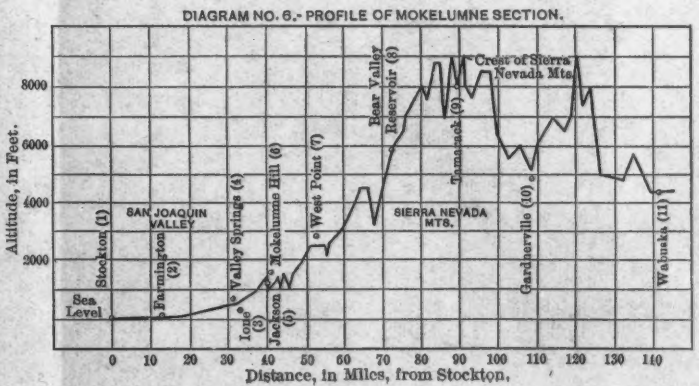


DIAGRAM NO. 6.- PROFILE OF MOKELUMNE SECTION.



TABOOSE GROUP OF PRECIPITA

DIAGRAM NO. 7.- RELATION OF ALTITUDE

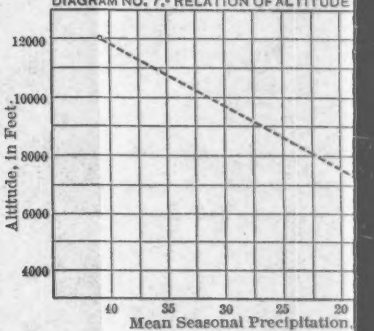


DIAGRAM NO. 8.

RELATION OF TOPOGRAPHIC LOCATION

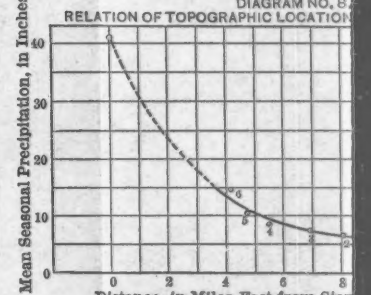
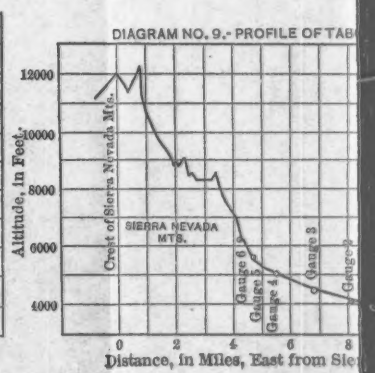


DIAGRAM NO. 9.- PROFILE OF TAB



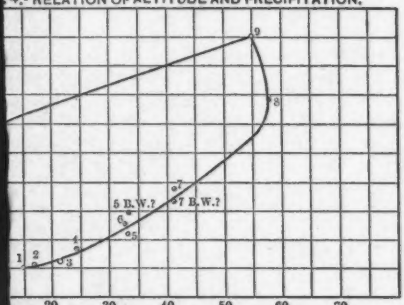
DIAGRAMS SHOWING THE

INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON PRECIPITATION.

AS ILLUSTRATED BY THE SIERRA NEVADA MTS.

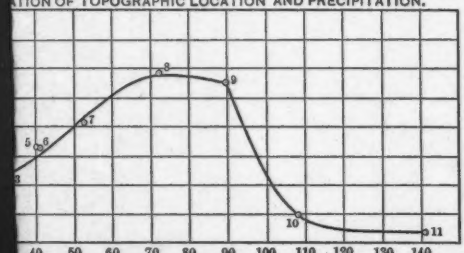
MNE GROUP OF PRECIPITATION GAUGES.

4.- RELATION OF ALTITUDE AND PRECIPITATION.



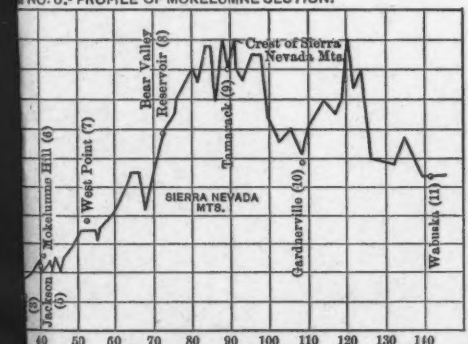
n Seasonal Precipitation, in Inches.

ATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.



Distance, in Miles, from Stockton.

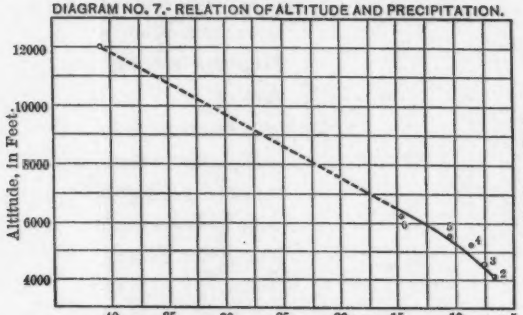
NO. 6.- PROFILE OF MOKELUMNE SECTION.



Distance, in Miles, from Stockton.

TABOOSE GROUP OF PRECIPITATION GAUGES.

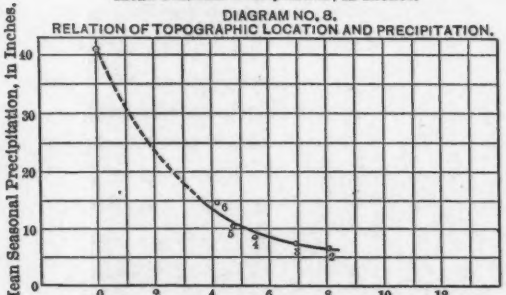
DIAGRAM NO. 7.- RELATION OF ALTITUDE AND PRECIPITATION.



Mean Seasonal Precipitation, in Inches.

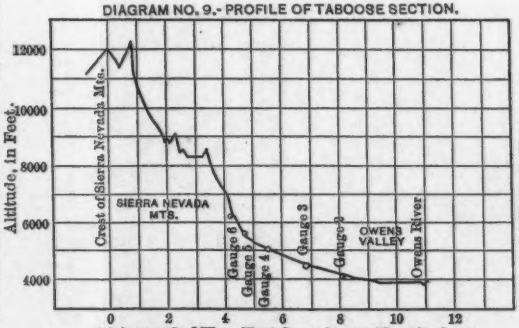
DIAGRAM NO. 8.

RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.



Distance, in Miles, East from Sierra Nevada Crest.

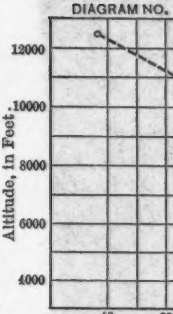
DIAGRAM NO. 9.- PROFILE OF TABOOSE SECTION.



Distance, in Miles, East from Sierra Nevada Crest.

OAK

DIAGRAM NO.



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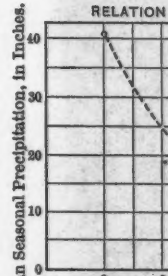
RELATION



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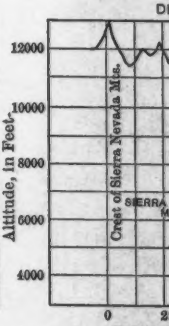

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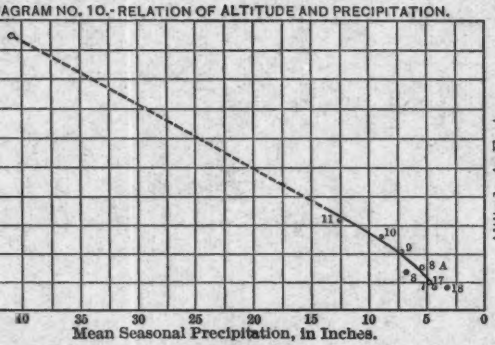
PLATE III.
PAPERS, AM. SOC. C. E.
APRIL, 1914.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

OAK GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 10.- RELATION OF ALTITUDE AND PRECIPITATION.



BAIRS GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 13.- RELATION OF ALTITUDE AND PRECIPITATION.

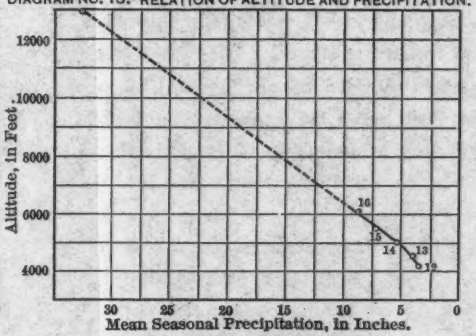


DIAGRAM NO. 11.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

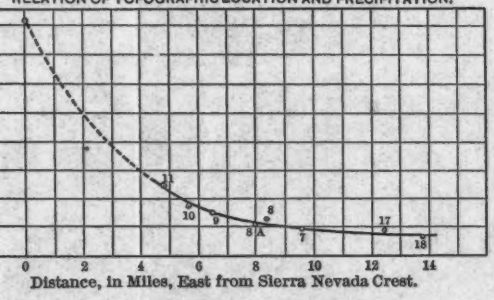


DIAGRAM NO. 14.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

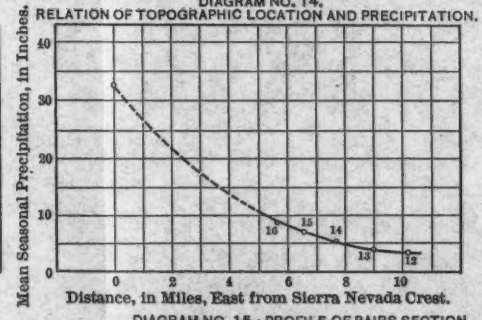


DIAGRAM NO. 12.- PROFILE OF OAK SECTION.

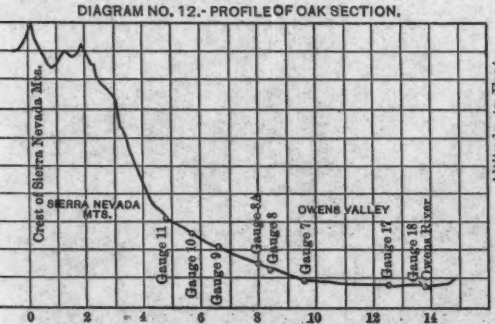
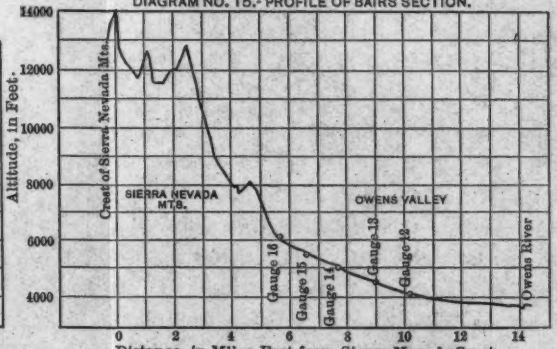


DIAGRAM NO. 15.- PROFILE OF BAIRS SECTION.



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India, and he developed for that region the empirical formula, $R = 1 + 1.92h - 0.40h^2 + 0.02h^3$, in which R represents the quantity of rain and h the relative height, in units of 1 000 ft., above an assumed plane which is itself 1 000 ft. above sea level. This equation, when platted, gives a curve very similar to that shown in Diagrams 1 and 4 of Plate III, the plane of maximum rainfall being 4 160 ft. above sea level. The equation does not apply to conditions on the leeward slope of a range, however, to judge by the discontinuity at the crest line shown on the Sierra Nevada curves. The straight-line relation between precipitation and elevation, which is often assumed in engineering computations, thus appears to have a very limited use, and to be at best a rough approximation.

The Los Angeles Aqueduct precipitation stations in Owens Valley lie in three groups, indicated on Fig. 1 and Plate II, as the Taboose, Oak, and Bairs Sections. The 2-year records for these stations were reduced to averages by comparison with the 26-year record at Independence. The platted curves for these sections (Plate III) are all similar in shape, and agree with the desert slope portion of the Central Pacific and Mokelumne curves. The highest point on each of the Owens Valley curves was obtained from the measured run-off of the canyons crossed by the section and a run-off factor chosen after careful study of precipitation and run-off data for Kings River, which drains the slope of the Sierra Nevada to the west. The precipitation at the mouth of the canyons was known from the rain gauge observations, and the average precipitation over each canyon from the run-off. Most of these canyon drainage areas are isosceles triangles in plan, the base being along the crest of the mountains. Also, judging by the Central Pacific and Mokelumne Sections, the precipitation increases uniformly from the base of the mountains on the desert side to the summit. Hence, the precipitation at the center of area of each drainage basin is equal to the average precipitation over the whole area, and the precipitation at the summit is obtainable by a simple proportion.

Preliminary to the establishment of isohyets or lines of equal annual precipitation for the Independence Basin, a broad study of precipitation was made for the whole Sierra Nevada range from Lake Tahoe to the Mojave Desert (Fig. 1). This was based on the California Water and Forest Association rainfall map of the State, pre-

pared in 1900, as revised between American and Kings Rivers by Edwin Duryea, Jr., M. Am. Soc. C. E., in 1908. With all available rainfall data to date, including the Los Angeles Aqueduct and several private records, the writer has made further revisions over the Southern Sierra and Owens Valley. The Water and Forest Association isohyets, as amended by Mr. Duryea, appear on Fig. 1 as solid lines, and revisions proposed by the writer are represented by dotted lines. Isohyets are shown with greater detail for the Independence Basin on Plate II.

Stream Flow.

Stream flow data essential to the determination of inflow and outflow for an underground reservoir, are the run-off from precipitation, the seepage from or into stream channels, and the flow of springs. Precipitation which finds its way into surface streams without absorption may, along some portion of the channel, percolate into porous gravels and join the subterranean supply. On the other hand, water may escape from the underground reservoir by seepage into stream channels where the general water plane is at a higher elevation than mean water level in the stream. Escape may also occur from springs.

The problem in the Independence Basin was, first, to classify the surface as to run-off characteristics and determine the run-off from each subdivision; second, to ascertain seepage losses from the seventeen tributary mountain streams between the canyon mouths and the valley floor; third, to determine the flow of springs which represent water escaping from the basin; and fourth, to ascertain whether Owens River made or lost water in passing through the basin.

Run-off.—It was early observed that the run-off characteristics of the four areas into which the region was classified for study (Table 1) were similar.

The clay soils of the valley floor occasionally yield a small run-off during and following winter precipitations of 1 in. or more in 24 hours, or warm rain falling on old snow. This water gathers and passes off into Owens River within a few hours by way of four waste channels. A study of the available data shows that the average total run-off from precipitation on the valley floor is about 2 sec-ft. of continuous flow.

The outwash slopes yield no appreciable surface run-off, on account of the porous gravel formation and the great depth to ground-

water. This fact has been established by repeated observations during and after rainstorms and thaws, and is confirmed by the noticeable absence of recent drainage channels or washes, except those of streams which derive their water from high mountain drainage areas.

The intermediate mountain slopes yield a small run-off during May and June, when the temperature at that level is sufficient to melt the accumulated winter snow, but the small streams do not advance far over the outwash slopes before they are entirely absorbed. If the precipitation of the preceding winter is below normal, the snow melts before the hot weather comes, and is absorbed at once. Springs are common along the lower borders of these slopes, the source being the melted snow absorbed by the porous material above and brought to the surface where it comes into contact with impervious formations. In only a few places does such water find its way into living streams.

The high mountain drainage areas have an abundant run-off, and perennial streams flow from all but one of them. The source of this water is precipitation, in the form of snow and rain, which falls within the drainage areas, and to a small extent snow dust carried over the summits by the prevailing west and northwest winds of winter and spring. For all practical purposes, the average discharge at the mouth of the canyon represents the average precipitation within the drainage area minus losses by evaporation from exposed snow surfaces. The underflow from these areas is negligible.

Stream discharge from the canyons is at a minimum from September to April. The flow during these months is remarkably uniform, and is entirely uninfluenced by the current storms, though from 70 to 80% of the annual precipitation occurs between November 1st and March 31st. The low-water flow is derived from springs and from the slow melting of the snow layer exposed to the earth's latent heat. Streams are usually frozen over by November, and as late as April they flow nearly to the mouths of the canyons in tunnels under the snow. Between April 1st and 20th air temperatures increase sufficiently to melt the snow at the lower elevations, and the streams begin to rise. There is an increase in air temperatures and stream flow from this date until the maximum flood crest is reached, some time between June 15th and July 15th, depending on the quantity of snow to be melted. Stream flow then decreases until some time in September,

after which low water prevails. About 70% of the annual run-off of the streams occurs during May, June, July, and August.

Percolation from Stream Channels.—The United States Geological Survey gauging stations on streams draining the high mountain areas are at the lower edges of the outwash slopes, just above the division boxes which apportion the water for use on the ranches of the valley floor. After leaving its canyon each stream traverses several miles of channel before reaching the gauging station, and preliminary observations in June, 1908, showed that considerable water (in some streams 50%) disappeared between the two points. It was necessary, therefore, either to establish regular gauging stations at the canyon mouths and depend on records for short periods, or to devise some means of computing the run-off from the high mountain areas from the existing Government records, which extended over 6 years. The latter method was chosen, and the results have proved very satisfactory.

The loss from these stream channels occurs as percolation into the porous alluvial material, direct evaporation from water surface, and transpiration from vegetation growing along the stream borders. Evaporation and transpiration losses were too small to be detected in current-meter work. As the expense of installing and maintaining weirs was prohibitive, the problem resolved itself into a study of percolation from stream channels.

There are three factors to be considered in a study of the subject: the rate of percolation, the area through which percolation occurs (the wetted perimeter), and the period of time during which a given unit of water is exposed (velocity of flow). The rate of percolation depends on (1) the character of the channel lining and the medium surrounding the channel, as regards size of pores and porosity; (2) the pressure gradient, depending on the difference in level of the surface of the water in the channel and the ground-water surface; and (3) the temperature of the water.

The effect of an increase in the wetted perimeter, other conditions being the same, is obviously to increase the percolation, but such change is accompanied by a proportionally larger increase in the velocity of flow, which reduces the time of exposure of a given volume of water. The net result, considering the total flow, is, therefore, a proportionally smaller percolation, although this effect may be counter-

acted to a certain extent by the scouring of a non-porous channel lining due to the increased carrying power of the stream. The whole matter is affected by so many indeterminate conditions that a general mathematical analysis is impossible, but, with these ideas in view, a study was made of each channel within the ordinary range of temperature and discharge.

The field work consisted of making comparative current-meter measurements at upper and lower stations on each creek, giving proper allowance of time for the passage of water between the two points. The measurements were made at intervals of from 6 weeks to 2 months, and extended over the period from June 15th, 1908, to September 15th, 1909, including the high-water periods of wet and dry seasons. Gauging sections were prepared at the mouth of the canyon on each creek. Estimates of the time required for the passage of water between stations were based on actual trial with aniline dye. Very little fluctuation in discharge was observed in any of the creeks between 8 A. M. and 5 P. M., even in the high-water period.

The temperature of the water as it issues from the canyons varies from 35° to 42° Fahr., in winter, and from 48° to 53° Fahr., in summer. In winter the temperature does not increase much as the water travels toward the valley. After leaving the protecting cover of the snow in the canyons during December and January the water actually becomes colder, ice prevailing for several weeks. In summer there is an average increase of 10° between the stations at the mouth of the canyon and in the valley.

Several methods were attempted for generalizing the results, but the most satisfactory was a graphical one, in which losses were platted as abscissas and stream discharges as ordinates with rectangular coordinates (Plate IV).

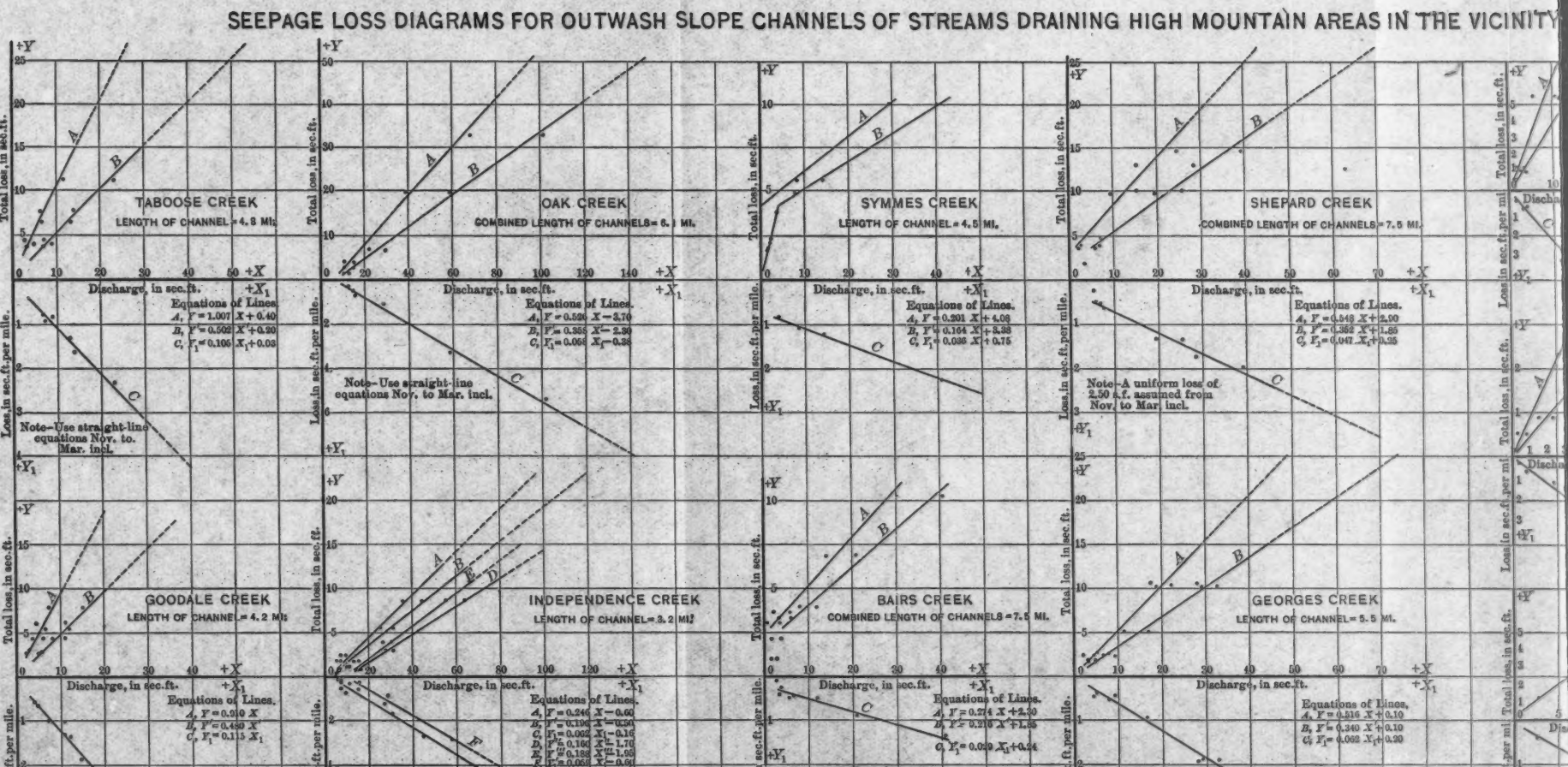
It was found that for each channel a straight line expressed the relation of these two quantities from April to October, inclusive. During the remaining 5 months, the relation is not clear, but the total loss is then so small that it can be obtained by inspection without affecting the accuracy of the computed discharge at the mouth of the canyon. Total losses are platted on the basis of discharge, both at lower and upper stations, so that, in correcting the Government records to obtain the true yield from high mountain drainage areas, the quantity desired can be obtained at once by entering on the X-axis

the discharge at the lower or Government station. It is also possible at the same time to read the loss, in second-feet to the mile, from the straight line below the X-axis.

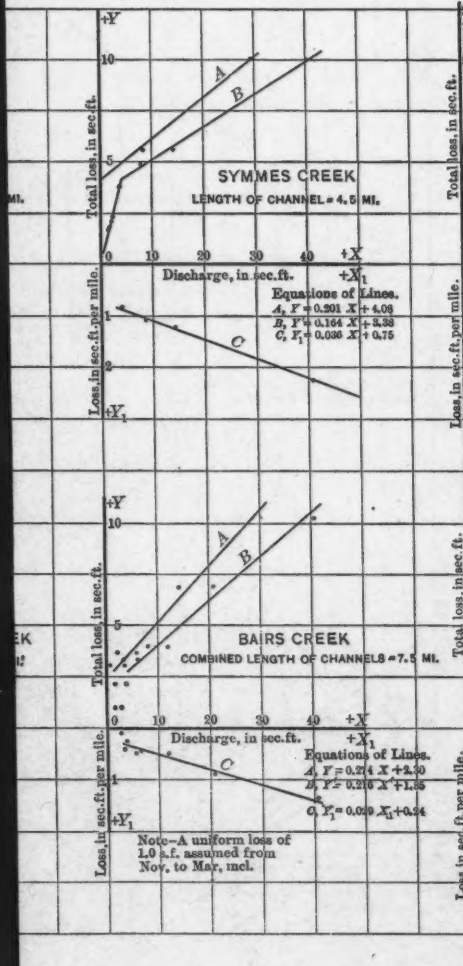
The diagrams on Plate IV were prepared for the purpose of computing (1) seepage losses above the U. S. Geological Survey gauging stations, and (2) the actual yield of high mountain drainage areas. In obtaining the field data, discharge measurements were made with small Price current meters during the period, June to August, 1909, inclusive, covering a season of small run-off and one of large run-off. The accuracy is up to the standard for a stream of this type. Measurements of medium and high stages were difficult, on account of the rough sections and very high velocities. The upper gauging station is at the mouth of the canyon, or below impervious dikes forcing seepage water to the surface. The average elevation is 6 000 ft. The lower station is that used by the U. S. Geological Survey (unless otherwise noted), which is above the diversion and has an average elevation of 4 100 ft. Below the latter there is ½ mile or more of channel suffering a large seepage loss which is not included. The length of the channel was obtained by scaling from the Mt. Whitney Quadrangle, and, for Taboose, Red Mountain, and Tinemaha Creeks, from a triangulation survey. Elevations were obtained in a similar manner. In using the diagrams, the straight-line relation applies from April to October, inclusive, during which time temperature and discharge vary similarly. The dotted portions are not supported by field data, and are to be used with judgment. From November to March, inclusive, temperature is the only effective variable, and arbitrary values for loss have been selected. Line "A" expresses the relation of total loss to discharge at the U. S. Geological Survey Station; Line "B" expresses the same relation at the mouth of the canyon; and Line "C" expresses the loss per mile to discharge at the mouth of the canyon. The diagrams are arranged so that all the four values involved may be obtained by entering any one of them.

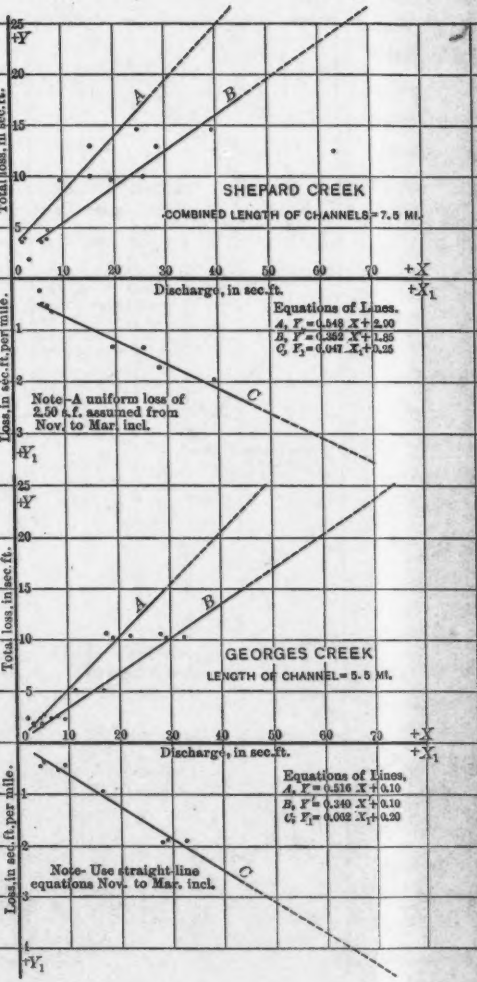
There are some interesting conclusions to be drawn from these diagrams. In general, the quantity of water percolating from the channels studied varies with the time of year and with different channel conditions. Variation with the time of year is due to the combined effects of temperature, area of wetted perimeter, and velocity of flow. These work more or less in harmony during April to October, inclusive.

SEEPAGE LOSS DIAGRAMS FOR OUTWASH SLOPE CHANNELS OF STREAMS DRAINING HIGH MOUNTAIN AREAS IN THE VICINITY



SLOPE CHANNELS OF STREAMS DRAINING HIGH MOUNTAIN AREAS IN THE





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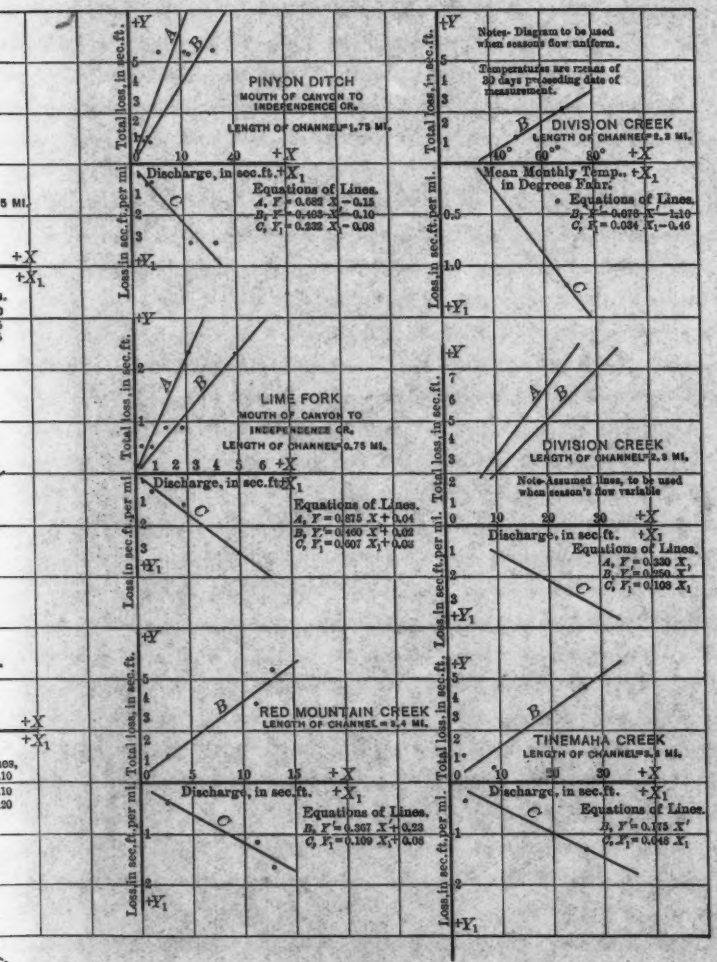

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and produce the straight-line relation of total loss and discharge. From November to March, inclusive, canyon discharges remain practically constant, showing that variations are largely controlled by temperature. Discharges are then so small, however, that errors of measurement are appreciable, and losses by evaporation have greater weight, so that the true relation of loss and temperature does not appear. A possible relation between total loss and temperature is suggested by the results on Division Creek, where the discharge at the mouth of the canyon was practically uniform during the period of study. Using as ordinates the mean air temperature at Independence for the 30 days preceding each date of measurement, we obtain a straight line which crosses the X-axis at about 35 degrees. A line supported by additional data might cross nearer 32°, the temperature at which percolation becomes physically impossible.

The character of the surrounding medium was the only channel condition which noticeably affected percolation. The loss from a channel crossing fissured lava, even where the lava was covered by a thin sheet of alluvium, was 30% greater than that in coarse alluvium. The streams studied do not overflow their channels, so that the effect of varying channel slopes and wetted perimeter could not be studied.

The run-off from each mountain canyon was computed from U. S. Geological Survey monthly mean discharge records, by use of the diagrams. The results are summarized in Tables 4 and 5. The missing seasons were estimated from the unbroken records of neighboring streams after a detailed study of yield per square mile and annual departure from normal for each stream. The long-term mean discharge was obtained by comparison with the Kings River record, which covered 21 years. This stream was chosen because its drainage area adjoined most of the Owens Valley streams and because conditions affecting run-off were more nearly similar than on any other stream. The results indicate that the total annual mountain run-off during the period of record was 153 annual sec-ft. and the normal 130 annual sec-ft. This total does not include Red Mountain and Tinemaha Creeks, which pass out of the basin after crossing a portion of the outwash slope. The normal run-off per square mile for streams north of the Kings-Kern divide with 60% or more of area above 10 000 ft., is 1.75 sec-ft., and, for streams similarly situated, with less than 60% above 10 000 ft., 1.18. For streams south of the Kings-Kern divide,

TABLE 4.—Seasonal Discharge, in Second-Feet, at United States Geological Survey Stations, of Creeks Tributary to Independence Region.

(Figures in parentheses are estimated.)

Creek.	Year Beginning September 1st.						Observed 5-year mean.	Computed 21-year mean.
	1904.	1905.	1906.	1907.	1908.	1909.		
Taboose	(5.7)	11.4	10.3	4.7	8.6	5.9	8.2	6.5
Goodale	(3.9)	5.4	6.6	3.8	6.4	5.3	5.5	4.3
Dry Canyon	0	0	0	0.0	0	0	0	0
Division	(3.9)	7.2	10.9	7.6	9.7	9.9	9.1	7.2
Sawmill	.....	5.4	(7.6)	(5.0)	7.3	7.2	6.5	5.1
Thibaut	(0.5)	(1.0)	(1.0)	0.8	0.9	0.2	0.6	0.5
Oak	(18.0)	31.8	23.9	15.8	30.8	18.4	24.1	19.0
Little Pine. } Pinyon }	(10.7)	28.5	22.5	11.8	25.8	17.2	21.2	16.7
Symmes	.....	(2.8)	3.1	0.8	6.3	1.1	2.8	2.2
Shepard	.....	23.1	11.0	7.2	12.9	7.7	12.5	9.8
Bairs	.....	8.0	4.8	2.0	6.1	2.4	4.7	3.7
George	.....	18.6	10.9	6.5	13.0	6.8	11.2	8.8
Hogback	(0)	(1.0)	(0.5)	(0)	(0.5)	0	0.4	0.3
	.....	144.2	113.1	66.0	128.3	82.1	106.7	84.0
Percentage of totals at mouth of canyon	.....	68	63	61	66	63	65	65

TABLE 5.—Seasonal Discharge, in Second-Feet, at Mouth of Canyon, of Creeks Tributary to Independence Region.

(Figures in parentheses are estimated.)

Creek.	Year Beginning September 1st.						Observed 6-year mean.	Computed 21-year mean.
	1904.	1905.	1906.	1907.	1908.	1909.		
Taboose	11.9	19.2	20.4	9.8	16.0	12.4	15.0	12.7
Goodale	7.6	9.9	12.3	7.2	11.2	10.2	9.8	8.3
Dry Canyon	(3.9)	(5.1)	(6.3)	(3.7)	(5.7)	(5.2)	5.0	4.2
Division	(4.4)	6.9	11.2	6.9	8.7	9.5	7.9	6.7
Sawmill	(4.0)	6.5	9.1	5.5	7.3	7.2	6.6	5.6
Thibaut	(1.9)	(3.1)	(4.4)	(2.7)	(3.5)	(3.4)	3.2	2.7
Oak	16.7	43.2	33.8	21.2	42.6	25.0	30.4	25.8
Little Pine	10.7	24.6	20.5	12.0	24.3	16.7	18.1	15.3
Pinyon	(3.6)	(8.9)	(6.6)	(4.2)	9.2	3.5	6.0	4.8
Symmes	(3.5)	(8.8)	7.0	3.7	10.2	4.9	6.2	5.3
Shepard	(11.1)	31.4	18.9	13.5	2.09	14.1	18.3	15.5
Bairs	(4.3)	11.8	7.7	4.2	9.4	4.8	7.0	5.9
George	(8.2)	23.8	16.5	9.8	18.7	10.3	14.6	12.4
Hogback	(2.8)	(7.8)	(5.7)	(4.2)	(6.8)	(3.7)	5.2	4.4
	94.6	211.0	180.4	108.6	194.5	130.9	153.3	129.6
Percentage reaching U. S. G. S. Stations.	....	68	63	61	66	63	65	65

the normal run-off is 1.36 and 0.86 sec-ft., respectively. It is also of interest to note that only 65% of this run-off reaches the Government gauging stations.

Springs.—The occurrence of springs in the basin is due to the reappearance of water which originally fell within its boundaries as precipitation and was absorbed. There are, in general, three types of springs which give rise to surface streams: those which derive their supply from precipitation on the intermediate mountain slopes, and appear at the base of these slopes; those which derive their supply from precipitation and stream percolation, and appear along the upper edge of the grass land; those which derive their supply from precipitation on lava flows, and appear at the lower borders of the flows.

The springs of the first type are not deep seated; they represent the drainage from the superficial deposits lying on the triangular mountain slopes between canyons. The temperature of their water is about 47° or 48° Fahr., and the flow in many of them increases in early summer and decreases during late summer and autumn. The water from most of these springs sinks into the porous gravels of the outwash slope, and joins the main body of ground-water in the basins.

The line of springs along the upper edge of the grass land represents the intersection of the natural surface of the ground and the surface of the ground-water. The water has penetrated rather deeply into the gravel fill, and issues with a temperature of about 62° Fahr., which is 5° higher than the mean annual temperature at Independence and 1° lower than that of water flowing from Artesian wells in the same location. The flow of these springs is variable, being least in late summer and greatest in early spring, with regular fluctuation between these dates, evidently depending on ground-water stages within the grass area. Only during the winter months is the discharge sufficient to be the source of surface streams which flow any considerable distance, and even then there are only a few of such streams which reach Owens River. Most of the yield of these springs is lost by evaporation and transpiration. The winter discharge of individual springs varies from 0.5 sec-ft. down to a quantity which is only enough to fill small pools of standing water, from which the evaporation equals the yield. The total winter discharge from all these springs is about 4 sec-ft.

The springs issuing from the lava formations are unique in having uniform discharges throughout the year and a temperature of 57° Fahr. The water is probably derived from precipitation on the lava surface, absorbed by the porous rock, and, by reason of the peculiar formation, gathered and delivered at the lower margin of the flow. The largest of these is Blackrock Springs (Plate II), 9 miles north of Independence. It has a discharge of 23 sec-ft., which flows out across the valley floor in two sloughs, each emptying into a series of shallow lakes. From November to March, inclusive, an average flow of about 7 sec-ft. reaches Owens River, but during the remainder of the year all the water is lost by seepage, evaporation, and transpiration. Hines Spring is 3 miles north of this spring, and has a continuous yield of about 4 sec-ft. Approximately, 1 sec-ft. finds its way into Owens River during the winter, but is lost during the remainder of the year. Campbell Spring is east of Owens River, 1 mile north of Aberdeen. It has a yield of about 0.5 sec-ft., and discharges directly into the river. Upper and Lower Seeley Springs are just above and just below Charlies Butte, and discharge directly into Owens River. The upper spring has a flow of 9.5 sec-ft., which is included in measurements of Owens River at the Butte. The lower one has a flow of 1.5 sec-ft.

Owens River.—Owens River flows lengthwise of the Independence Basin for 29 miles, although the actual length of its channel is possibly 20% greater, owing to its sinuosity. It is the drainage outlet for the waste surface water of the region, including the run-off from the valley floor, the yield of springs, and a small portion of the run-off from high mountain drainage areas. In order to account for all escaping surface waters, and determine the condition of the river channel with regard to seepage, observations of river discharge were made daily near the north and south boundaries of the region, and measurements of discharge into and diversion from the river channel were made between these two points. Complete data are available for 1909 and 1910. Analysis of these data shows that seepage losses occur during high-water, and seepage gains during low-water stages. The net result is a loss between Charlies Butte and Whitney Bridge, which can be accounted for by channel evaporation. The water plane of the valley on each side of the river lies between high- and low-water levels in the river. Hence, seepage gain and loss are the result of local ab-

sorption and drainage along the river channel, and have no relation to the general ground-water situation of the basin.

Evaporation and Transpiration.

Evaporation from Water Surfaces.—Measurements of evaporation from free water surfaces were made under three conditions: from a pan floating in a body of water, from a pan placed in the soil, and from a deep tank placed in the soil. The first and second were designed to furnish data regarding evaporation from reservoir surfaces and from areas of shallow flood water, respectively. The third was desired for purposes of comparison with records of evaporation from soil. The pans, which were of the pattern used by the U. S. Reclamation Service, were 3 ft. square and 10 in. deep, and were of galvanized sheet-iron. Observations were made by replacing the quantity evaporated with a cup having a capacity equal to a depth of 0.01 in. in the pan. The initial height of the water surface was such that a pin, projecting from the center of the pan and remaining at a fixed height, 2 in. below the rim, was just submerged. The deep tank was circular, 3½ ft. in diameter and 4 ft. deep, and observations were made in a stilling well with a hook-gauge and vernier scale reading to 0.01 in. The records were all kept near Independence, and observations were made every second day in summer and every fourth day in winter.

The record for the pan in water (Table 6) is available from August 4th, 1908, to June 1st, 1911. The pan, Fig. 3, at first, was in Blackrock Slough, but was moved to its final location, in Owens River at Citrus Bridge, on May 7th, 1909. The pan was supported by a timber float, which protected it from splashing water. The depth of water beneath the pan varied from 1 to 5 ft., depending on the river stage. The river water had a moderate velocity, and varied in temperature from about 75° Fahr., in summer, to about 40° Fahr., in winter. The river banks averaged 4 ft. high above the water surface, and the pan was about 30 ft. from them. Rain gauge No. 18 was 100 ft. away, on the river bank, and was observed in connection with the evaporation record.

The record for the pan in soil is broken. It extends from August 1st, 1909, to November 30th, 1909, and from March 14th, 1910, to June 1st, 1911. The pan, Fig. 4, was in the valley floor at the soil evaporation experiment station, about 3 miles east of Independence.

It was set in a shallow excavation with soil banked up to about half the depth of the pan. Water temperatures range from 95° Fahr., in summer, to 32° Fahr., in winter. The surface temperature was about 1° warmer than that for the mixed contents of the pan. Table 7 summarizes the results by months for this pan, and, by comparing it month by month with the evaporation from the pan in water, an average excess of about 33% is observed. This is probably due to the higher temperature of the water in the pan in soil during the hours of sunlight.

TABLE 6.—Depth of Evaporation, in Inches, From Water Surface Near Independence (Pan in Water).

Month.	1908.		1909.		1910.		1911.		Average percentage of annual evaporation.
	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	
January	.....	.....	1.60	0.052	1.75	0.056	1.65	0.053	2
February	.....	.....	2.40	0.086	2.50	0.089	2.35	0.084	4
March	*	.....	4.70	0.152	5.15	0.166	3.70	0.119	7
April	.....	.....	7.30	0.243	7.05	0.235	6.25	0.208	11
May	.....	.....	9.60	0.310	8.29	0.267	8.01	0.258	13
June	.....	.....	10.10	0.337	9.90	0.330	.....	.....	15
July	.....	.....	10.40	0.335	8.50	0.274	.....	.....	14
August	* 4.90	0.222	8.00	0.258	8.20	0.264	.....	.....	12
September	5.30	0.176	6.60	0.220	6.30	0.210	.....	.....	10
October	3.50	0.113	3.90	0.126	4.20	0.135	.....	.....	6
November	2.50	0.083	2.60	0.087	2.36	0.079	.....	.....	4
December	1.50	0.048	(1.85)	0.060	1.24	0.040	.....	.....	2
	.....	.....	69.05	0.189	65.44	0.179	.....	.....	100

* August 10th to 31st, inclusive.

The deep-tank record extends unbroken from April 16th, 1909, to Dec. 31st, 1911. The tank, Fig. 6, was at the soil evaporation experiment station, and was set in the soil with the upper rim flush with the surface. The water surface was not allowed to fall more than 4 in. below the rim. The temperature of the surface water varied from 80° Fahr., in the heat of summer to freezing in winter. Except during freezing weather, the average temperature of the contents of the tank was 5° less than that of the surface layer. The presence of the surrounding soil makes the range in temperature less than that for the shallow pan. The record, which is presented in Table 8, indicates an annual depth of evaporation practically equal to that from the pan in water at Citrus Bridge. The monthly distribution is more uniform,

TABLE 7.—DEPTH OF EVAPORATION, IN INCHES, FROM WATER SURFACE NEAR INDEPENDENCE (PAN IN SOIL).

Month.	1909.			1910.			1911.		
	Total.	Rate per 24 hours.	Percentage of evaporation from pan in water.	Total.	Rate per 24 hours.	Percentage of evaporation from pan in water.	Total.	Rate per 24 hours.	Percentage of evaporation from pan in water.
January	.....	.....	.....	.....	.....	.....	2.25	0.073	138
February	.....	.....	.....	.....	.....	.....	2.25	0.080	95
March	.....	.....	.....	*4.25	0.236	.....	4.80	0.155	130
April	.....	.....	.....	9.50	0.316	135	8.12	0.271	130
May	.....	.....	.....	10.61	0.342	128	10.25	0.330	128
June	.....	.....	.....	11.95	0.398	121	.....	.....	.....
July	.....	.....	.....	12.55	0.405	148	.....	.....	.....
August	10.70	0.345	134	11.80	0.381	144	.....	.....	.....
September	8.50	0.283	129	8.80	0.293	140	.....	.....	.....
October	5.80	0.187	149	5.60	0.180	133	.....	.....	.....
November	3.80	0.127	146	2.85	0.095	121	.....	.....	.....
December	.....	.....	.....	1.60	0.052	129	.....	.....	.....
	.....	.....	.....	.....	.....	133	.....	.....	.....

*March 14th to 31st, inclusive.

TABLE 8.—DEPTH OF EVAPORATION FROM WATER SURFACE NEAR INDEPENDENCE.

DEEP TANK IN SOIL.

Month.	1909.			1910.			1911.			Percentage of annual evaporation during each month.
	Total, in inches.	Rate, in inches per 24 hours.	Percentage of evaporation from pan in water.	Total, in inches.	Rate, in inches per 24 hours.	Percentage of evaporation from pan in water.	Total, in inches.	Rate, in inches per 24 hours.	Percentage of evaporation from pan in water.	
Jan.	.....	.....	.....	2.00	0.064	114	2.30	0.074	139	3
Feb.	.....	.....	.....	2.90	0.104	116	2.55	0.091	108	4
Mar	.....	.....	.....	5.60	0.180	109	3.95	0.127	107	7
Apr	2.90*	0.193	.....	7.40	0.246	105	6.80	0.226	84	10
May	7.50	0.242	78	7.71	0.248	93	7.90	0.254	77	12
June	7.80	0.260	77	8.60	0.287	87	6.65	0.222	.....	11
July	7.90	0.254	76	8.30	0.268	98	8.60	0.277	.....	12
Aug	8.20	0.264	102	8.80	0.284	107	9.65	0.311	.....	14
Sept.	7.20	0.240	109	7.30	0.243	116	7.16	0.239	.....	11
Oct.	5.00	0.161	128	5.15	0.166	123	4.90	0.158	.....	7
Nov.	3.30	0.110	127	3.10	0.103	131	3.00	0.100	.....	5
Dec.	(2.20)	0.071	119	2.15	0.069	173	(2.50)	0.081	.....	4
Totals	.....	.....	.....	69.01	0.188	106	65.96	0.180	....	100

*For period, April 16th to 30th, inclusive.

there being 70% of the total during the 6 summer months and a difference of 27 in. between summer and winter evaporation. The effect on evaporation of the modified temperature extremes of the soil is well shown by comparison with the record for the pan in water (Table 6). The temperature conditions for the deep tank agree closely with those of the surrounding soil.

Evaporation from Ground Surface.—Water in the surface layers of the ground is subject to evaporation, either directly from the soil or through vegetation by the process of transpiration. It is available for evaporation in Owens Valley under two conditions: temporarily, following a rainstorm or sudden thaw, and permanently, within areas where the average depth to ground-water does not exceed 8 ft. The total evaporation under the first condition is relatively unimportant, because of the infrequency of storms and the small quantity of precipitation, and no attempt was made to measure it. Under the second condition, however, evaporation losses are large, for, not only is soil capillarity able to draw gravity water to the surface, but roots of vegetation, such as wild grass, penetrate the soil to ground-water and become the channels by which a large quantity of moisture is conveyed into the atmosphere. Evaporation from bare soil combined with transpiration is, in fact, the most important element entering into computations relating to ground-water for this region. So few data are available on the subject that extended observations were undertaken.

Owens Valley is an ideal location for carrying on such experiments. In the first place, the source of water available for evaporation may be kept under the complete control of the observer as regards the quantity and rate of supply. Storms are rare, and the total precipitation is small, so that little uncertainty exists from this cause regarding the quantity of percolation from precipitation on the surface of a body of isolated soil. Second, the method by which the surface soils of the valley floor are kept moist can be reproduced artificially on a small scale with only a slight departure from natural conditions. The source of supply for soil moisture is a permanent ground-water surface from which water is drawn by capillary forces. This ground-water is replenished by percolation from the precipitation and surface water of the intermediate mountain and outwash slopes, which seeps laterally toward the valley floor and lies beneath it under hydrostatic pressure sufficient to maintain a permanent ground-water surface.

FIG. 6.—DEEP-WATER EVAPORATION TANK.

FIG. 7.—SOIL TANK NO. 3 IN OPERATION.

Similar pressure can be reproduced in the bottom layer of an isolated body of soil, and capillary forces can be depended on to raise moisture to the surface. Finally, the large annual depth of evaporation makes possible a more accurate determination of its quantity than in a less arid region. Experiments carried on under these conditions have been very satisfactory.

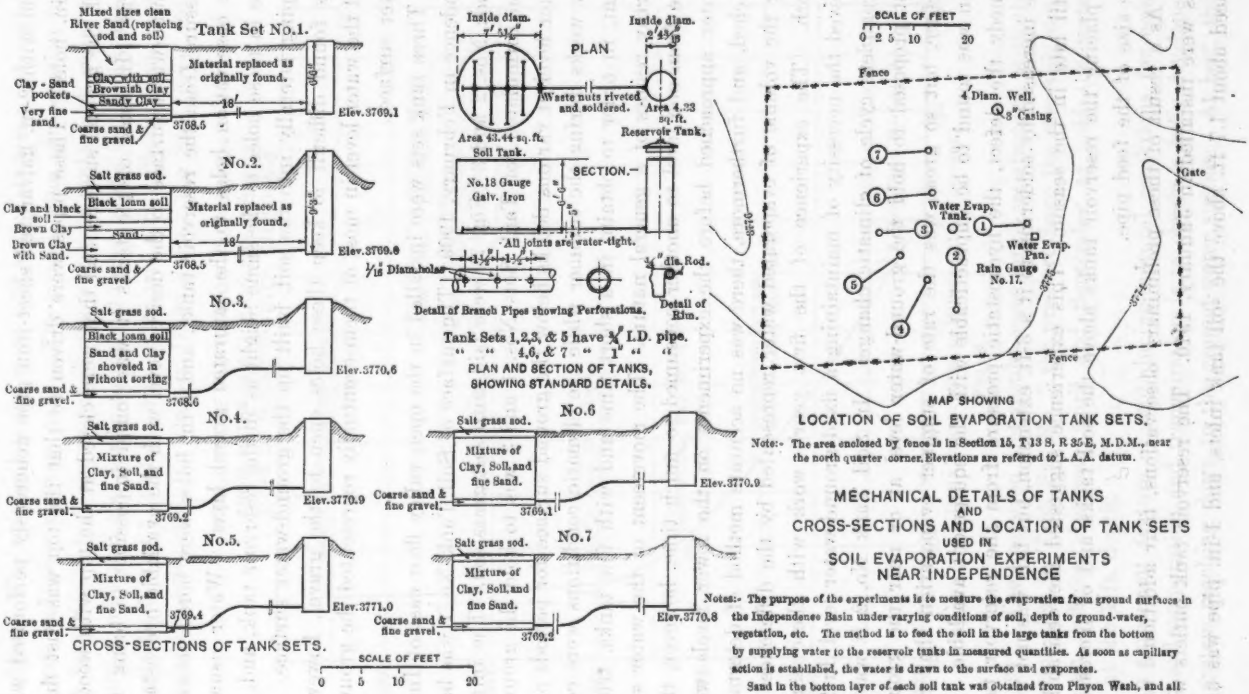
The rate of evaporation from soil depends on the temperature of the air and soil, the quantity of moisture already in the immediately surrounding atmosphere, the quantity of moisture in the surface layers of the soil, and the character of the vegetation and other soil covering. The first two of these factors have the same effect on soil evaporation as on that from a free water surface—higher air and soil temperatures result in increased evaporation, as does also dryer atmosphere or increased movement of wind. The third factor is directly proportional to the rate of evaporation, because the loss of moisture occurs from soil grains at or very near the surface. The quantity of moisture in the soil available for evaporation thus depends on the character of the soil, as regards capillarity and depth to the ground-water surface. For example, in a coarse, sandy soil, "gravity water" will be drawn to the surface through the capillary spaces from depths not exceeding 4 ft., and, in a fine sandy or clayey soil, water will be drawn from depths as great as 8 ft. The last factor, the extent and character of vegetation, affects the evaporation rate both through the activity of transpiration and the effect on capillarity. Plant roots are continually absorbing water from the soil; this water passes off into the atmosphere through the leaves, and the evaporation losses from soil are greatly increased thereby. The roots of native salt grass will penetrate to a depth of 8 ft. in search of water. A further effect of the growth of vegetation is to increase the vertical capillary flow of moisture through soil by way of the many tubes filled with the rotted fiber of dead roots. These tubes are the result of years of growth, and penetrate the soil in all directions above the ground-water surface.

The purpose of the experiments was to obtain data sufficiently complete to compute the total volume of water annually lost by evaporation and transpiration from the valley floor. This involved making observations under the various local conditions which affect soil evaporation. The plan was to reproduce natural conditions in isolated bodies of typical soil and determine the evaporation therefrom for

varying climatic conditions, depths to ground-water, soils, and vegetation.

The experimental equipment consists of two galvanized-iron tanks, 6½ ft. in depth, connected at the bottom by an 18-ft. length of galvanized pipe. (See Fig. 8.) The smaller tank is 2 ft. $4\frac{3}{16}$ in. in diameter, and has a tight-fitting cover. The larger tank is 7 ft. 5¼ in. in diameter, and has a system of branching perforated pipes at the bottom connected with the pipe from the smaller tank. The two tanks and all connections are water-tight, and water poured into the smaller or reservoir tank passes into the larger or soil tank and escapes through the perforations. These two tanks were placed in excavations of proper size to receive them, the soil tank was filled with the excavated soil, and the reservoir tank was filled with water. A 6-in. layer of screened gravel, too coarse to enter the $\frac{1}{16}$-in. perforations, was laid in the bottom of the soil tank in order to insure an uninterrupted and well-distributed feeding of water from the reservoir tank into the superimposed soil. As soon as the material became saturated and capillary action was established to the surface, the water level in the soil was brought to the desired depth and kept there by supplying water to the reservoir tank in measured quantities. Volumetric measurements of water poured into or withdrawn from the reservoir tanks were made with an ordinary gallon measure. Accumulation or depletion of the supply in the reservoir tank was determined volumetrically by measuring the depth of water with a steel tape. The volume passing out of the reservoir tank during a given period represents the total evaporation from the soil tank during that period.

The position of the ground-water surface in the soil tank was determined by measuring its depth below the ground surface in 2-in. augur holes bored in the soil to a proper depth. Measurements were made from a fixed point with a steel tape weighted at the end and chalked before each observation. Three holes were placed in each tank, half way between the center and rim, on radii 120° apart. The holes were not bored deep enough to reach the bottom layer of coarse gravel, and the water level in them represented the ground-water surface in the surrounding soil. An average of the observations made at a given time was assumed to represent the general depth to ground-water for the tank at that time. The tendency of the sides of the holes to cave in and the bottom to fill with sand was controlled by casing



Note:- The area enclosed by fence is in Section 15, T 13 S, R 35 E, M.D.M., near the north quarter corner. Elevations are referred to L.A.A. datum.

MECHANICAL DETAILS OF TANKS AND CROSS-SECTIONS AND LOCATION OF TANK SETS USED IN SOIL EVAPORATION EXPERIMENTS NEAR INDEPENDENCE

Notes:- The purpose of the experiments is to measure the evaporation from ground surface in the Independence Basin under varying conditions of soil, depth to ground-water, vegetation, etc. The method is to feed the soil in the large tanks from the bottom by supplying water to the reservoir tanks in measured quantities. As soon as capillary action is established, the water is drawn to the surface and evaporates.

Sand in the bottom layer of each soil tank was obtained from Pinyon Wash, and all sizes less than 1/16" were screened out.

FIG. 8.

them with 2-in. galvanized sheet-iron pipe generously perforated with $\frac{1}{16}$-in. holes. These pipes were driven so that the top was just flush with the ground surface, and they were closed at the top with wooden plugs. In some of the tanks it was found impossible to bring the ground-water surface to the desired level with the available hydrostatic pressure from the reservoir tanks, and 2-in. holes were bored between the observation holes to the saturated gravel layer. Water usually rose in these holes to the same height as in the reservoir tank, and, by seeping laterally into the soil, built up the ground-water surface. It was found difficult to keep these holes open to the gravel, however, and the water level in most of them eventually represented the ground-water surface.

Three tank sets were installed in the open valley floor east of Independence in February, 1909. The surface of Soil Tank No. 1 was bare sand; Nos. 2 and 3 (Fig. 7) were laid with salt-grass sod. The initial plan formulated for Tank Sets Nos. 1 and 3 was to hold the ground-water level at various depths below the ground surface for periods of a few weeks during the summer while the climatic conditions were constant, in order to obtain, in a short time and with few tanks, trustworthy results of a general nature. The movement of the water surface from one level to another consumed so much time, however, that winter approached before the experiments on the lower levels were reached, and furthermore, there was no accurate method of determining the volume of evaporated water represented by the differences in depth. The experience of the first year's work with these tanks showed the necessity of maintaining a fixed ground-water level during a complete cycle of climatic changes. In Soil Tank No. 2 it was at first proposed to hold the ground-water level at or near the ground surface, but so great was the rate of summer evaporation that this plan was found to be impracticable with the equipment available. To remedy the defect, the hydrostatic pressure from the reservoir tank was increased by soldering to it a 3-ft. extension, but this was not used until late in the season. This experience suggested the desirability of placing the reservoir tanks above the soil tanks and of increasing the size of the feed pipe.

As a result of these preliminary observations, four additional tank sets were installed in January, 1910. The reservoir tank outlets were placed about 1.7 ft. above the soil tank inlets, and 1-in. pipe was used

throughout. The new soil tanks were laid with salt-grass sod, which took root and grew in every tank. The general plan of operation for Tank Sets Nos. 2 to 7 was to supply the reservoir tanks with water in quantities such that the depths to ground-water in the soil tanks were, respectively, 5 ft., 4.5 ft., 4 ft., 3 ft., 2 ft., and 1 ft. Observations were carried on continuously on the six tanks during the two years, 1910

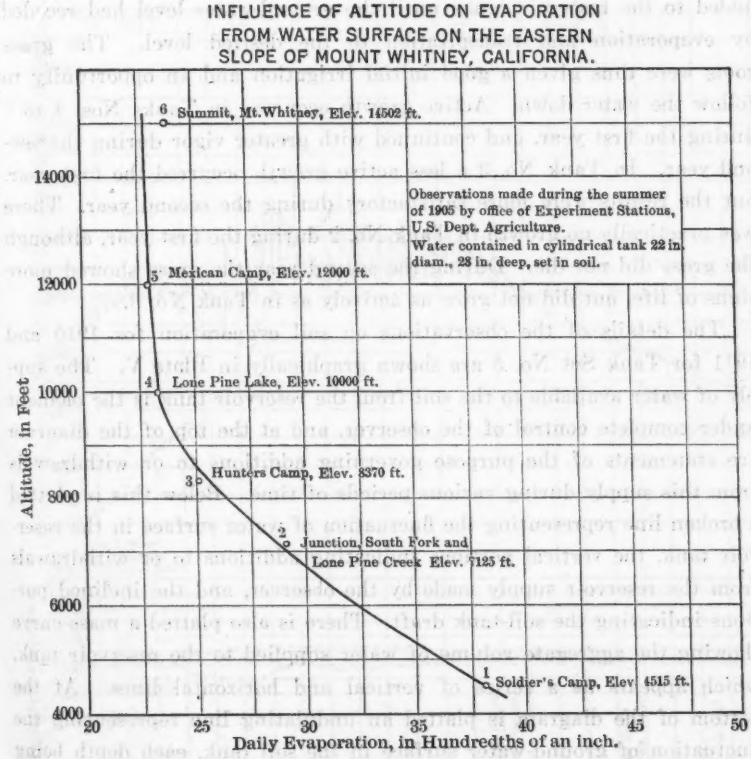


FIG. 9.

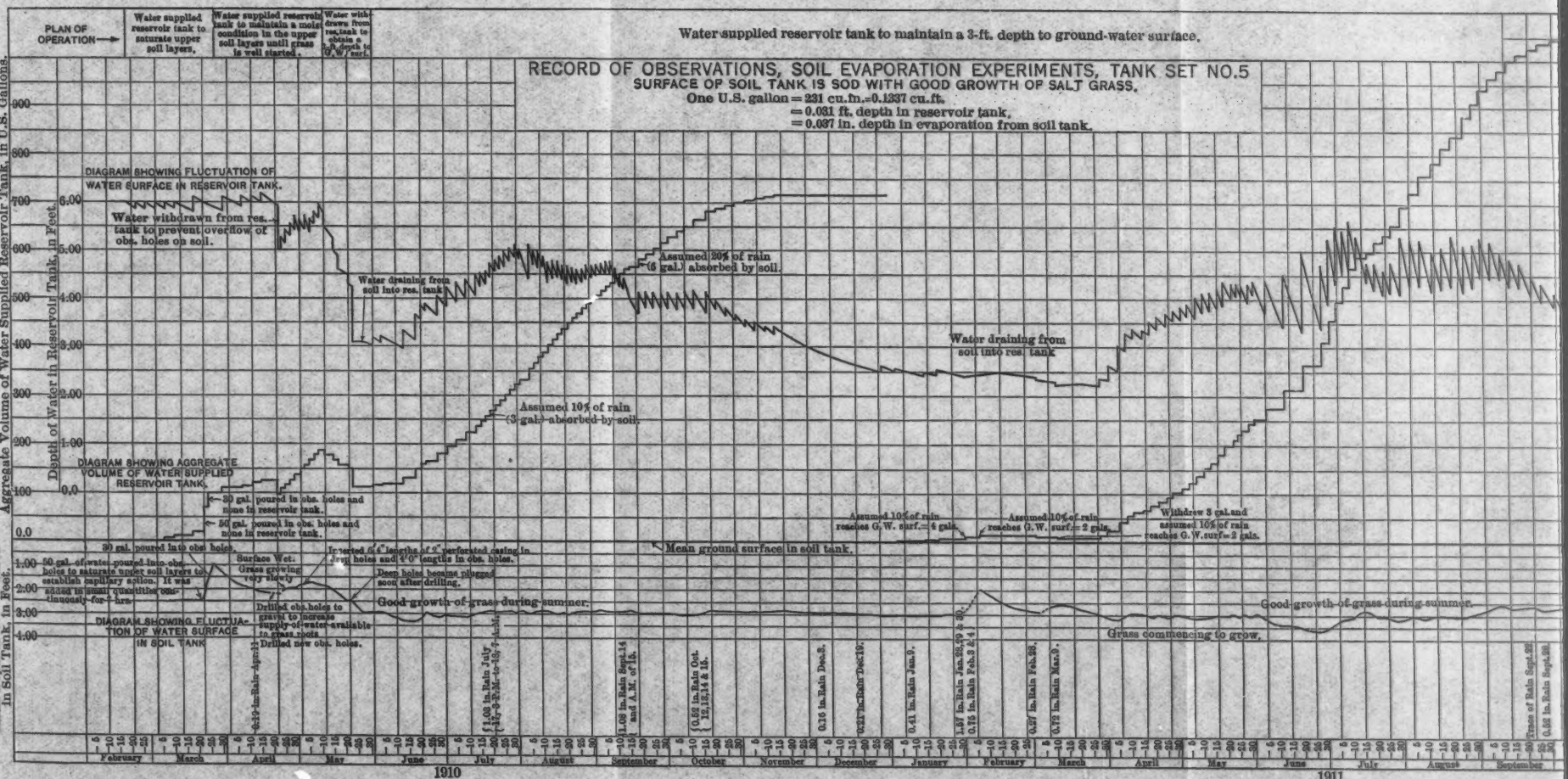
and 1911. The operation and results were quite satisfactory, with the exception that in Tank Set No. 7 the pressure from the reservoir tank was not sufficient between April and September to hold the water level at the 1-ft. depth.

An important feature of reproducing natural conditions for combined soil evaporation and transpiration is to obtain a fully developed root system reaching down to the ground-water surface. At best, this

requires more than a year, particularly for the greater depths. In order to stimulate the growth as much as possible, the water level in all soil tanks was brought up to about 1 ft. below the surface as soon after installation as possible. This was accomplished by pouring water into the observation holes until the soil was completely saturated to the level desired and the surface showed moisture. Then no water was added to the reservoir tanks until the ground-water level had receded by evaporation and transpiration to the desired level. The grass roots were thus given a good initial irrigation and an opportunity to follow the water down. Active growth occurred in Tanks Nos. 4 to 7 during the first year, and continued with greater vigor during the second year. In Tank No. 3 a less active growth occurred the first year, but the results were more satisfactory during the second year. There was practically no growth in Tank No. 2 during the first year, although the grass did not die. During the second year the grass showed more signs of life, but did not grow as actively as in Tank No. 3.

The details of the observations on soil evaporation for 1910 and 1911 for Tank Set No. 5 are shown graphically in Plate V. The supply of water available to the soil from the reservoir tank is the element under complete control of the observer, and at the top of the diagram are statements of the purpose governing additions to or withdrawals from this supply during various periods of time. Below this is platted a broken line representing the fluctuation of water surface in the reservoir tank, the vertical portions indicating additions to or withdrawals from the reservoir supply made by the observer, and the inclined portions indicating the soil-tank draft. There is also platted a mass-curve showing the aggregate volume of water supplied to the reservoir tank, which appears as a series of vertical and horizontal lines. At the bottom of the diagram is platted an undulating line representing the fluctuation of ground-water surface in the soil tank, each depth being obtained by averaging the depths recorded in the observation holes.

The small part that precipitation plays in ground-water fluctuations in Owens Valley is shown by this diagram. The average annual precipitation at the experiment station is about 4.38 in., the season, 1909-10, being normal and 1910-11 well above normal. At the bottom of the diagram is noted the date and quantity of precipitation for each storm. It is seen that, even in a wet season, percolating water does not penetrate to depths exceeding 2.5 ft., unless more than 1 in. falls


PLAN OF OPERATION
Water supplied reservoir tank to saturate upper soil layers.
Water supplied reservoir tank to maintain a moist condition in the upper soil layers until grass is well started.
Water withdrawn from res. tank to obtain a 3-ft. depth to G.W. surf.
Water supplied reservoir tank to maintain a 3-ft. depth to ground-water surface.
RECORD OF OBSERVATIONS, SOIL EVAPORATION EXPERIMENTS, TANK SET NO.5
SURFACE OF SOIL TANK IS SOD WITH GOOD GROWTH OF SALT GRASS.
One U.S. gallon = 231 cu.in.=0.1337 cu.ft.
= 0.031 ft. depth in reservoir tank,
= 0.037 in. depth in evaporation from soil tank.
Aggregate Volume of Water Supplied Reservoir Tank, in U.S. Gallons.
Depth of Water in Reservoir Tank, in Feet.
Mean Depth to G.W. Surface in Soil Tank, in Feet.
DIAGRAM SHOWING FLUCTUATION OF WATER SURFACE IN RESERVOIR TANK.
Water withdrawn from res. tank to prevent overflow of obs. holes on soil.
Water draining from soil into res. tank
Assumed 20% of rain (6 gal.) absorbed by soil.
Assumed 10% of rain (3 gal.) absorbed by soil.
Water draining from soil into res. tank
DIAGRAM SHOWING AGGREGATE VOLUME OF WATER SUPPLIED RESERVOIR TANK.
30 gal. poured in obs. holes and none in reservoir tank.
50 gal. poured in obs. holes and none in reservoir tank.
30 gal. poured into obs. holes.
50 gal. of water poured into obs. holes to saturate upper soil layers to establish capillary action. It was added in small quantities continuously for 7 hrs.
Surface Wet.
Grass growing very slowly
Inverted 6" lengths of 2" perforated casing in open holes and 2'0" lengths in obs. holes.
Deep holes become plugged soon after drilling.
Drilled obs. holes to gravel to increase supply of water available to grass roots.
Drilled new obs. holes.
DIAGRAM SHOWING FLUCTUATION OF WATER SURFACE IN SOIL TANK
Mean ground surface in soil tank.
Good growth of grass during summer.
Grass commencing to grow.
Good growth of grass during summer.
Assumed 10% of rain reaches G.W. surf. = 4 gals.
Assumed 10% of rain reaches G.W. surf. = 2 gals.
Withdrew 3 gal. and assumed 10% of rain reaches G.W. surf. = 2 gals.
0.12 in. Rain April 1.
1.00 in. Rain July 15-16, 7 A.M. to 7 A.M.
1.08 in. Rain Sept. 14 and A.M. of 15.
0.52 in. Rain Oct. 12, 13, 14 & 15.
0.15 in. Rain Dec. 8.
0.12 in. Rain Dec. 19.
0.41 in. Rain Jan. 9.
1.37 in. Rain Jan. 28, 29 & 30.
0.76 in. Rain Feb. 3 & 4
0.37 in. Rain Feb. 23.
0.72 in. Rain Mar. 9.
Times of Rain Sept. 22.
0.52 in. Rain Sept. 26.
1910
1911

Water supplied reservoir tank to maintain a 3-ft. depth to ground-water surface.

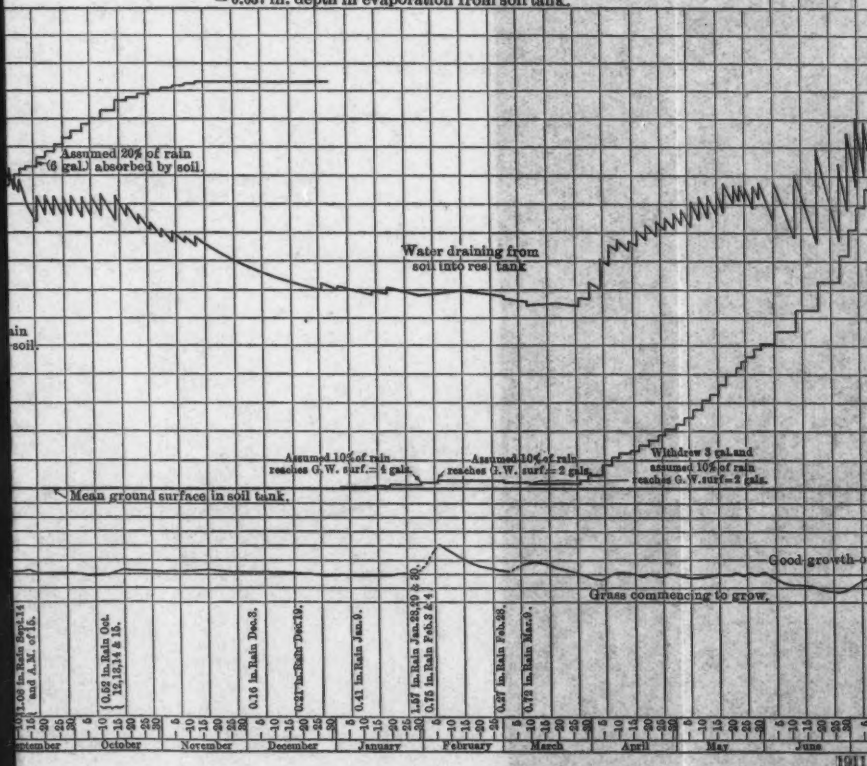
OF OBSERVATIONS, SOIL EVAPORATION EXPERIMENTS, TANK SET NO.5

SURFACE OF SOIL TANK IS SOD WITH GOOD GROWTH OF SALT GRASS.

One U.S. gallon = 231 cu.in.=0.1337 cu.ft.

= 0.031 ft. depth in reservoir tank.

= 0.037 in. depth in evaporation from soil tank.



1911

PLATE V.
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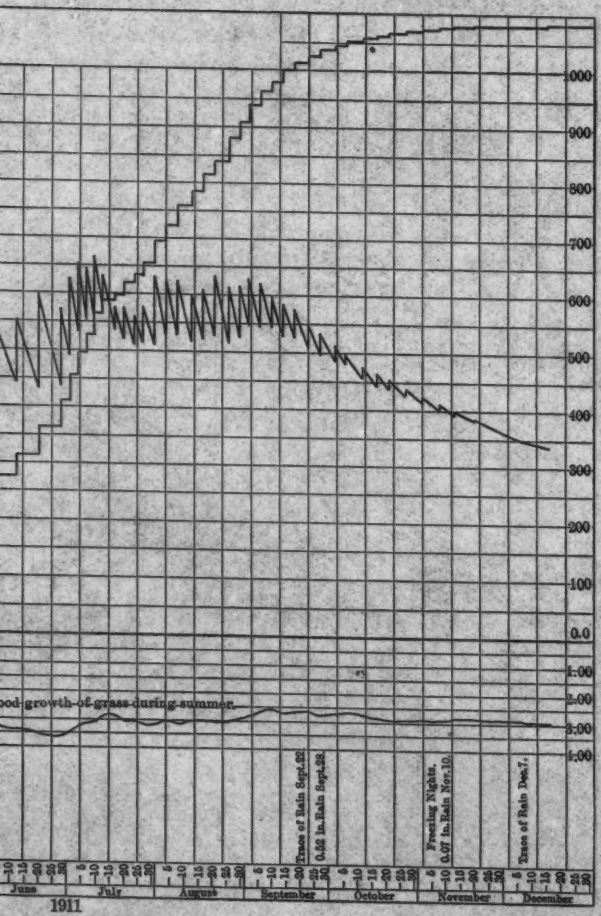
YIELD OF UNDERGROUND RESERVOIRS.



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within a short period on moist soil. Even then it does not appear to reach depths greater than 4 ft. The problem of percolation from rainfall, therefore, is practically eliminated from the experiments. When rising ground-water was noted in a soil tank after precipitation, the volume of percolating water was estimated from the observed rise and included in the mass-curve, as noted on the diagrams.

The quantity of water evaporated from the soil surface of any tank during a given period can be computed accurately from the diagrams, when the depth to ground-water at the beginning and end of the period is the same, by noting from the mass-curve the quantity supplied to the reservoir tank during the period and the accumulation or depletion in the reservoir tank. The sum of these quantities, with their proper algebraic sign, gives the loss by evaporation. For differing depths to ground-water, however, the computations are only approximate, because the proportion of empty space in the soil layer and the quantity of moisture it contained initially are both unknown. A monthly summary of results for Tank Sets Nos. 2 to 7 for 1911 is presented in Tables 9 to 14. The annual depth of evaporation from the several soil tanks exhibited a consistent decrease with increase of depth to ground-water, and varied from 48.8 in. for No. 7 to 13.43

TABLE 9.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 2.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan.	6	1.10	1.15	+ 2	4	0.15	0.005	4.98
Feb.	0	1.15	1.11	− 1	1	0.04	0.001	4.95
Mar.	3	1.11	1.14	+ 1	2	0.07	0.002	4.94
Apr.	10	1.14	1.18	+ 1	9	0.33	0.011	4.94
May	30	1.18	1.19	0	30	1.11	0.036	4.98
June	66	1.19	1.39	+ 6	60	2.22	0.074	5.03
July	71	1.39	1.22	− 5	76	2.81	0.091	5.00
Aug.	105	1.22	1.78	+ 18	87	3.22	0.104	4.95
Sept.	52	1.78	1.52	− 8	60	2.22	0.074	4.80
Oct.	18	1.52	1.37	− 5	23	0.85	0.028	4.80
Nov.	1	1.37	1.18	− 6	7	0.26	0.009	4.85
Dec.	1	1.18	(1.10)	− 3	4	0.15	0.005	4.98
Year.	363	1.10	1.10	0	363	13.43	0.037	4.94

TABLE 10.—Depth of Evaporation From Ground Surface Near Independence During 1911.

Tank Set No. 3.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan.....	9	0.14	0.10	— 1	10	0.37	0.012	4.53
Feb.....	18	1.10	0.23	+ 4	14	0.52	0.019	4.59
Mar.....	12	0.23	0.20	— 1	13	0.48	0.015	4.48
Apr.....	40	0.20	0.54	+ 11	29	1.07	0.036	4.50
May.....	62	0.54	0.60	+ 2	60	2.22	0.072	4.49
June.....	103	0.60	0.96	+ 12	91	3.37	0.112	4.63
July.....	160	0.96	0.67	— 9	169	6.25	0.202	4.51
Aug.....	285	0.67	2.00	+ 43	242	8.96	0.289	4.52
Sept.....	135	2.00	1.13	— 28	163	6.03	0.201	4.22
Oct.....	27	1.13	0.54	— 19	46	1.70	0.055	4.21
Nov.....	4	0.54	0.16	— 12	16	0.59	0.020	4.33
Dec	3	0.16	(0.10)	— 2	5	0.18	0.006	4.56
Year.....	858	0.14	0.10	0	858	31.74	0.087	4.46

TABLE 11.—Depth of Evaporation From Ground Surface Near Independence During 1911.

Tank Set No. 4.

Month.	Volume of water supplied to reservoir tank in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate in inches per 24 hours.	
Jan.....	7	1.53	1.49	— 1	8	0.30	0.010	3.97
Feb.....	— 8	1.49	1.18	— 10	2	0.07	0.002	3.36
Mar.....	3	1.18	1.19	0	3	0.11	0.004	3.74
Apr.....	62	1.19	2.15	+ 31	31	1.15	0.038	4.00
May.....	96	2.15	2.57	+ 14	82	3.04	0.098	4.06
June.....	121	2.57	2.63	+ 2	119	4.40	0.147	(3.34)
July.....	114	2.63	1.97	— 21	135	5.00	0.161	3.44
Aug.....	141	1.97	2.47	+ 16	125	4.63	0.149	3.92
Sept.....	65	2.47	1.90	— 18	83	3.07	0.102	3.85
Oct.....	30	1.90	1.53	— 12	42	1.55	0.050	3.93
Nov.....	9	1.53	1.09	— 14	23	0.85	0.028	3.97
Dec.....	6	1.09	(0.91)	— 6	12	0.44	0.015	4.10
Year.....	646	1.53	0.91	— 19	665	24.61	0.067	3.81

TABLE 12.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 5.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to groundwater surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan	8	2.55	2.43	— 4	12	0.44	0.014	3.01
Feb	6	2.43	2.42	0	6	0.22	0.008	2.47
Mar	3	2.42	2.36	— 2	5	0.18	0.006	2.73
Apr	88	2.36	3.65	+ 42	46	1.70	0.057	2.99
May	141	3.65	4.09	+ 14	127	4.70	0.151	3.01
June	166	4.09	4.19	+ 3	163	6.03	0.201	3.40
July	244	4.19	4.14	— 2	246	9.11	0.294	3.06
Aug	255	4.14	4.92	+ 23	232	8.58	0.280	2.99
Sept	127	4.92	4.07	— 25	152	5.62	0.188	2.69
Oct	36	4.07	3.24	— 27	63	2.33	0.075	2.77
Nov	9	3.24	3.67	— 18	27	1.00	0.033	2.83
Dec	1	2.67	(2.45)	— 7	8	0.30	0.010	2.90
Year	1 084	2.55	2.45	— 3	1 087	40.21	0.110	2.90

TABLE 13.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 6.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to groundwater surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan	12	3.03	3.19	+ 5	7	0.26	0.008	1.92
Feb	9	3.19	3.17	— 1	10	0.37	0.013	1.31
Mar	24	3.17	3.32	+ 5	19	0.70	0.023	1.70
Apr	94	3.32	3.53	+ 7	87	3.22	0.107	2.00
May	163	3.53	3.54	0	163	6.03	0.195	1.99
June	182	3.54	3.75	+ 7	175	6.48	0.216	2.30
July	213	3.75	3.31	—14	227	8.40	0.271	2.02
Aug	314	3.31	4.25	+30	284	10.50	0.339	2.01
Sept	143	4.25	3.89	—12	155	5.74	0.192	1.55
Oct	38	3.89	3.36	—17	55	2.04	0.065	1.65
Nov	8	3.36	2.98	—12	20	0.74	0.025	1.84
Dec	6	2.98	(2.83)	— 5	11	0.41	0.013	2.04
Year	1 206	3.03	2.83	— 7	1 213	44.89	0.122	1.86

TABLE 14.—Depth of Evaporation From Ground Surface Near Independence During 1911.

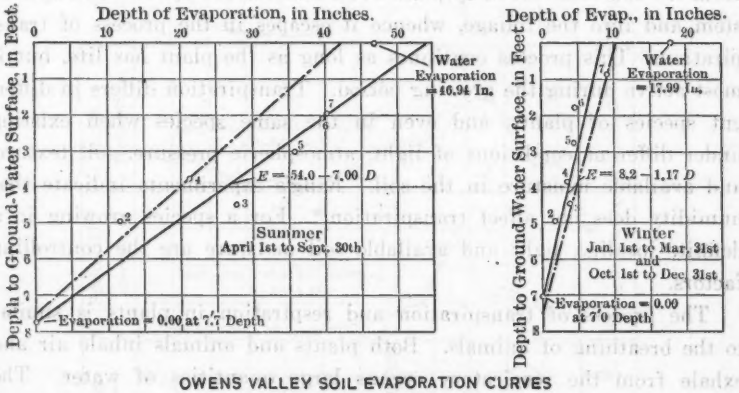
Tank Set No. 7.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan.....	3	5.07	4.70	— 12	15	0.56	0.018	0.73
Feb.....	31	4.70	5.08	+ 12	19	0.70	0.025	0.81
Mar.....	67	5.08	5.90	+ 26	41	1.52	0.049	0.98
Apr.....	102	5.90	5.96	+ 2	100	3.70	0.123	1.46
May	151	5.96	5.92	— 1	152	5.62	0.181	1.64
June.....	160	5.92	5.62	— 10	170	6.30	0.210	2.06
July.....	223	5.62	5.40	— 7	230	8.52	0.275	2.19
Aug.....	255	5.40	5.99	+ 19	236	8.74	0.281	2.51
Sept.....	178	5.99	5.79	— 6	184	6.81	0.227	2.39
Oct.....	115	5.79	5.77	— 1	116	4.29	0.139	1.44
Nov.....	14	5.77	4.96	— 26	40	1.48	0.049	0.60
Dec.....	0	4.96	(4.50)	— 15	15	0.56	0.018	0.60
Year.....	1 299	5.07	4.50	— 18	1 318	48.80	0.133	1.45

in. for No. 2. The depth of summer evaporation varied from 81 to 90% of the annual in the several tanks, and averaged 87 per cent. The month of maximum evaporation is August, and minimum evaporation occurs during December to March, inclusive. The exact date of maximum evaporation rates for the several soil tanks occurs about September 1st, and they follow each other consecutively with greater depth to ground-water. The approximate dates of maximum and minimum air temperatures at Independence are July 10th and January 10th, respectively, but no measurements were made to determine the lag of corresponding soil temperatures at various depths. The extremes of evaporation from water surfaces agree in time of occurrence with maximum and minimum air temperatures, however, and the observed lag in soil evaporation is in general consistent with the observed lag in soil temperatures in other localities. Hence it is reasonable to conclude that extremes in the rate of soil evaporation and soil temperature are concurrent at a given depth.

A graphic study of the data in Tables 9 and 14 for the periods, April 1st to September 30th, and October 1st to March 31st, which are, respectively, periods of increasing and decreasing evaporation rate, is presented in Fig. 10. There appears to be, during each period,

a straight-line relation between total evaporation and depth to ground-water. The limiting depth is 7.7 ft., and the total evaporation when water and ground surface coincide, 54.0 and 8.2 in., respectively. The total depth of evaporation, in inches, being represented by E, and the depth to ground-water, in feet, by D, the equations representing variation in evaporation with depth to ground-water are $E = 54.0 - 7.00\ D$ and $E = 8.2 - 1.17\ D$. It will be noted that the combined soil evaporation and transpiration during the summer exceeded the water evaporation from the tanks in the soil by 15 per cent.



OWENS VALLEY SOIL EVAPORATION CURVES
FOR ALKALI SALT GRASS LAND
Based on Soil Tank Observations of 1911.
——— Curve based on 1911 Data.
– – – – Curve based on 1910-11 Data.
o 1911 Observations.

FIG. 10.

The curve based on the 1910-11 data is shown on the diagram for comparison. There was a marked increase in total evaporation during 1911, as compared with the season, 1910-11, which was due to the more complete development of the root systems. Broadly speaking, the results for 1911 showed an increase of 17% in the volume of water consumed during the two periods. A continuation of observations for another year would show still further increase for those tanks in which the grass roots had not reached the water plane. Observations of depth to water in test holes in the transition zone between meadow and desert land indicate that soil evaporation ceases for depths exceeding 8 ft. The effect of increased evaporation for the tanks with greatest depth to ground-water would be to drop the lower end of the

curve to some point below 7.7 ft. The true curve, therefore, is probably steeper than that for 1911, crossing the X-axis at about the same point, but the Y-axis at about 8 ft. instead of 7.7 ft. However, in the practical use of the curve, the departure of the lower end from the true position does not affect materially the computations of the total volume of water evaporating from a given area, as the proportion of such volume originating in areas of relatively deep ground-water is small.

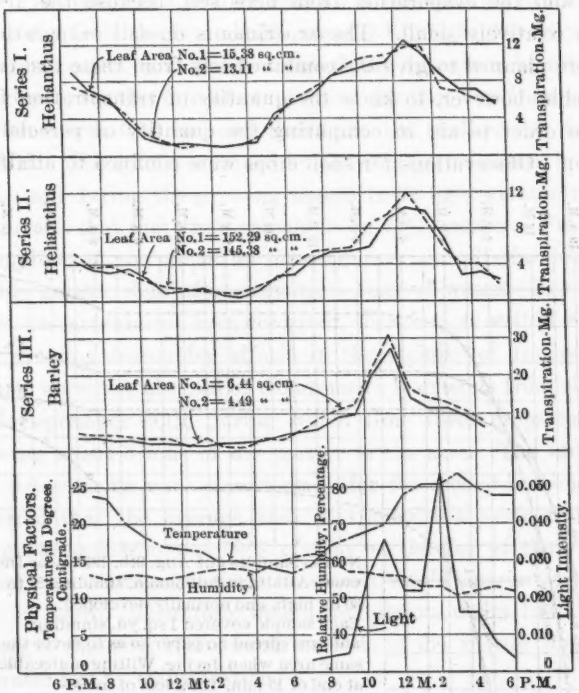
Transpiration.—A considerable portion of the water evaporating from the soil is absorbed by plant roots and carried upward through the stem and into the foliage, whence it escapes in the process of transpiration. This process continues as long as the plant has life, but is most active during the growing period. Transpiration differs in different species of plants, and even in the same species when existing under different conditions of light, atmospheric pressure, soil texture, and available moisture in the soil. King's experiments indicate that humidity does not affect transpiration.* For a species growing in a definite locality, light and available soil moisture are the controlling factors.

The process of transpiration and respiration in plants is similar to the breathing of animals. Both plants and animals inhale air and exhale from the respiratory organs large quantities of water. The lungs of animals are intended primarily to provide a means for the entrance of oxygen into the body and for the escape of carbon dioxide, but they cannot perform their functions unless the interior lining of the air cells is kept moist. Similarly, the breathing surface of a plant must be kept moist, and, as a protection from too rapid evaporation, this surface is within the plant structure, principally in the foliage. Plant leaves are enclosed in a relatively impervious skin or epidermis in which are small breathing pores or stomata which open or close automatically, depending on the needs of the plant for a greater or less quantity of air. When exposed to light, the food-manufacturing processes of a green plant are stimulated, and require a continually changing volume of air in contact with the breathing surface. The stomata open proportionally to the light intensity. Should the water supply in contact with the roots be insufficient, the breathing surface may become dry, and when that happens the stomata close automatically

* "Irrigation and Drainage," by F. H. King, New York, 1899.

until the proper quantity of air is admitted for the plant to do its work under the new conditions. The stomata, therefore, control the quantity and rate of loss of water from plants by transpiration.

There is a marked diurnal periodicity in the rate of transpiration, which investigators are led to believe is largely the result of the varying intensity of light. This periodicity is well illustrated by ob-



RELATION OF TRANSPIRATION TO LIGHT INTENSITY DURING A 24-HOUR DAY.

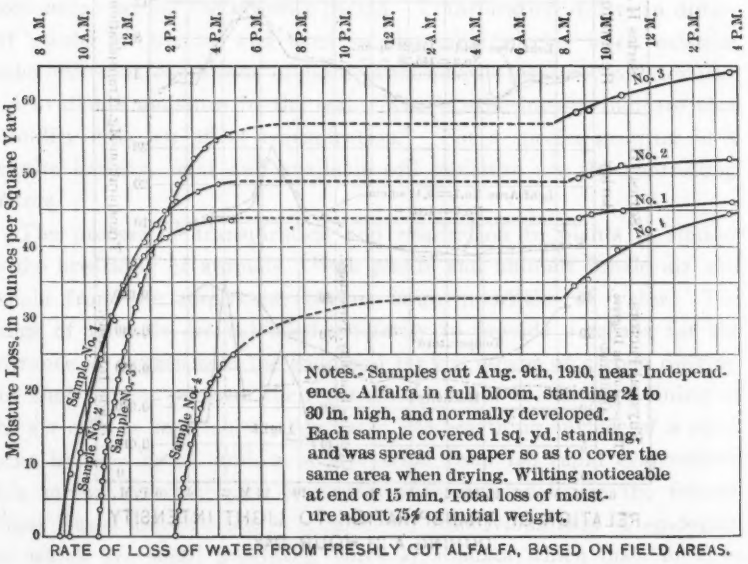
FIG. 11.

servations made under the direction of Mr. Frederick E. Clements, State Botanist of Minnesota, and reproduced in Fig. 11.* Measurements of transpiration were made hourly from 6 P. M. on February 16th to 6 P. M. on February 17th, and the physical factors were observed between these hours. The day was cloudy throughout, so that

* "Influence of Physical Factors on Transpiration", by A. W. Sampson and L. M. Allen, Minnesota Bot. Studies, Pt. I, Vol. 4, 1909, p. 42.

the variation in temperature and humidity was slight. The diagrams show very strikingly the response of transpiration to changes in intensity of light.

No measurements of transpiration are available for conditions similar as regards altitude and aridity to those in Owens Valley. It is unnecessary in this study to know separately the transpiration from wild grasses and the evaporation from bare soil, because the area of the latter is relatively small. The experiments on soil evaporation, therefore, were planned to give the combined loss from these two causes. It is desirable, however, to know the quantity of transpiration from field crops, in order to aid in computing the quantity of percolation from irrigation. Observations for such crops were confined to alfalfa.



RATE OF LOSS OF WATER FROM FRESHLY CUT ALFALFA, BASED ON FIELD AREAS.

FIG. 12.

The method of measurement was based on the assumption that the rate of loss of water from freshly cut plants would correspond closely to the rate before cutting. The plants were cut rapidly from a measured area, weighed, and spread out on paper to cover the same area as before cutting. At short intervals they were reweighed until there was no further appreciable loss. No noticeable wilting occurred during the first 15 min. and the rate of loss during this period was used as

a basis for calculations. The results of the measurements are shown graphically in Fig. 12.

The rapid decrease in the rate of loss is very noticeable. Inspection of Fig. 11 will show that the rates of transpiration at 8.45, 9.15, and 10.30 A. M., and 2.02 P. M., expressed as percentages of the average rate for 24 hours, are, respectively, 128, 141, 177, and 197, an average of 161 per cent. If a similar relation is assumed, the average loss in a 24-hour day from the four alfalfa samples would be 366 oz. per sq. yd. of field area, or 0.49 in. in depth. This figure appears to be rather large, at first glance, for the rate of evaporation for that day from the pan in Owens River was 0.30 in., and that from the shallow pan in the soil was 0.38 in. The results obtained by German investigators indicate the loss from sod during the growing season to be 92% greater than from water surface, and that from cereals to be 73% greater. Furthermore, the humidity of the air after passing over an alfalfa field is very noticeably greater than after crossing a body of water. The result obtained in the experiment here described, therefore, is within reason.

The growing season for alfalfa in the vicinity of Independence is marked by an entire absence of cloudiness. It extends from about April 15th to September 30th, during which time three crops mature, the yield being about 5 tons of dry matter to the acre. The samples used for the experiments were almost ready for the second cutting. On the assumption that the average area of transpiring surface during the entire growing season was 50% of that on the day of the experiment, the total loss of water during the season would amount to 41 in., or 3.43 ft. in depth. Therefore, with a production of dry hay, amounting to 5 tons to the acre, there would be 1 lb. of dry matter for every 93.5 lb. of water lost by transpiration.

Ground-Water.

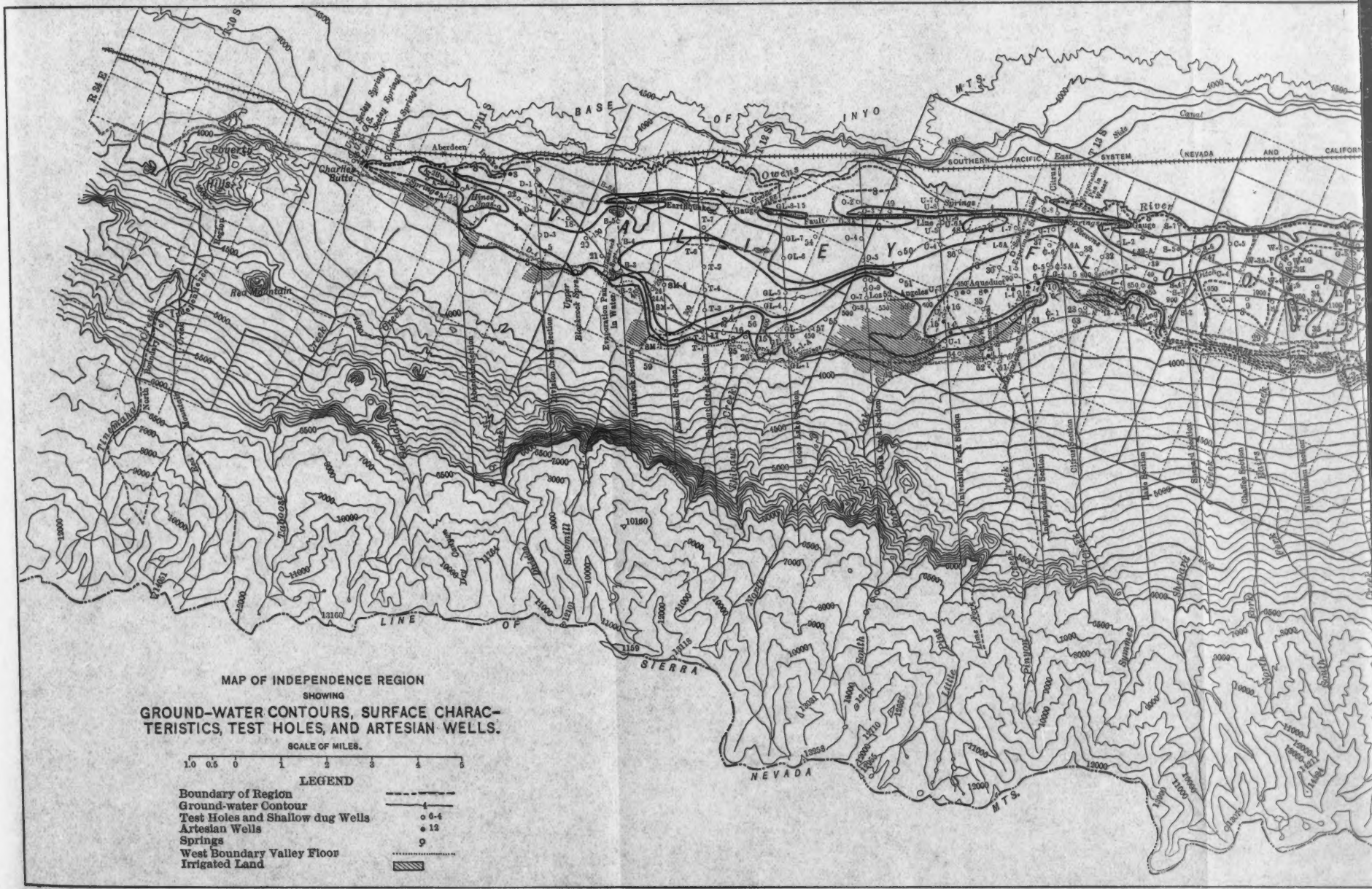
Form of the Ground-Water Surface.—The general form of the ground-water surface corresponds with that of the surface of the valley fill, although the slopes are less steep and the irregularities are not so pronounced. In the valley floor the depth to ground-water is only a few feet. It becomes progressively greater toward the mountains, and probably lies 200 or 300 ft. beneath the outwash slope at about the 5 000-ft. contour. Superimposed on the general ground-water surface are sharp "ridges" beneath stream channels and "mounds" under irrigated

fields. The surface of the water in the underground reservoir, therefore, is not a level plain, but has a varied topography.

There are two reasons for this condition: the action of gravity tending to equalize inequalities in the ground-water surface, and the resistance which the ground offers to the lateral motion of water through its interstices. Percolating waters enter the valley fill from the upper edge of the outwash slope, from stream channels crossing the outwash slope, and from irrigated fields. The valley floor is the lowest portion of the valley fill and also the ground-water outlet. The force of gravity, therefore, tends to draw percolating water which has reached the surface saturation to the level of the valley floor. This can occur only by a lateral movement of water from the outwash slope toward the valley floor, but the resistance of the porous material is so great that a steep gradient is necessary to maintain even a very low velocity. Hence there is the steep slope of the ground-water surface from the mountains toward the valley, at many points exceeding 80 ft. per mile, and laterally from stream channels and irrigated fields. The lateral movement of the water is so slow that percolating water entering at the upper edge of the outwash slopes, does not reach the valley for at least 2 years.

In order to outline the ground-water definitely, all existing domestic wells in the region were located and many additional observation wells were drilled, where the cost was not prohibitive. The region contains 27 domestic wells, 12 of which are on the valley floor and 15 on the outwash slopes. There were drilled in addition 142 observation wells, all but two being on the valley floor. These wells were sufficient to define the ground-water surface over about 60 sq. miles of the region.

Ground-water contours for the valley floor showing lines of equal average depth to ground-water have been worked out on Plate VI. They represent the average position of the surface of saturation between the annual extremes. The data are sufficient to determine the 3-, 4-, and 8-ft. contours with reasonable accuracy. The sudden approach of ground-water toward the surface at the upper edge of the grass land is shown, and also the general proximity of ground-water to the surface throughout the valley floor. The total area between the westerly 8-ft. contour and Owens River is 67 sq. miles. The average depth to ground-water is between 4 and 8 ft. over 40% of this area, and between 3 and 4 ft. over 28 per cent. It exceeds 8 ft. over 14% of the area, and is 3 ft. or less over 18 per cent. The area of the valley floor is 66.4 sq. miles


R.34 E.
R.35 E.
T.12 S.
T.13 S.
BASE OF INYO MTS.
SOUTHERN PACIFIC East SYSTEM (NEVADA AND CALIFORNIA)
Canal
Aberdeen
Owens River
Earthquake
Fault
Springs
Gauge
Lone Pine
Diversion Dam
Los Angeles Aqueduct
Intake
Power House
Charlies Butte
Red Mountain
Haiwee Reservoir
Upper Haiwee Spring
Lower Haiwee Spring
Haiwee Creek
Rose Valley
Rose Spring
Rose Valley Spring
Cottonwood Spring
Cottonwood Creek
Cottonwood Canyon
Cottonwood Section
Olancha Section
Olancha Creek
Olancha Peak
Olancha Cone
Olancha Sand Dunes
Olancha Dunes
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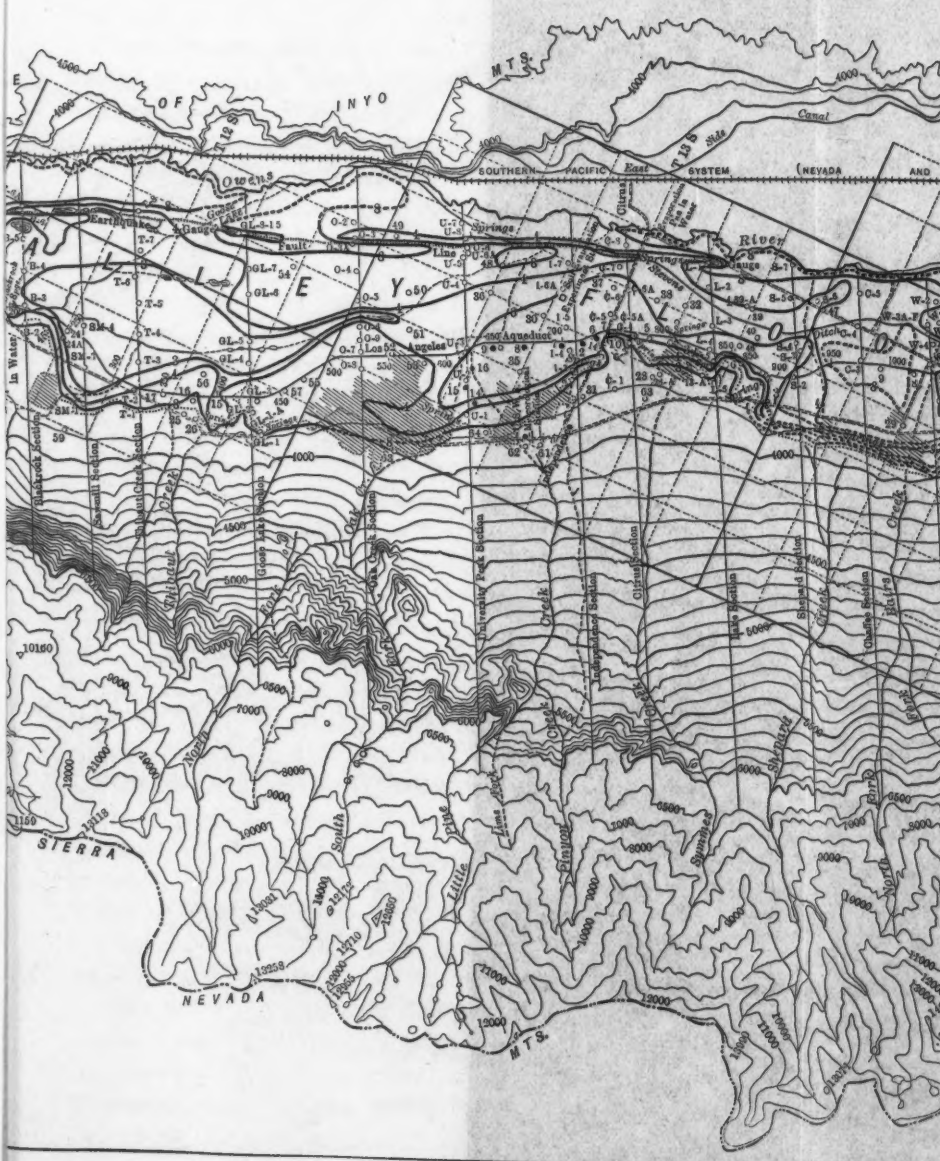
Olancha Dunes Spring Canyon


OF INYO
MTS.
Canal
SOUTHERN PACIFIC
East SYSTEM
(NEVADA
AND
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Earthquake
Gauge
Fault
Springs
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Gauge
Aqueduct
Los
Angeles
Pitch
In Water
Creek
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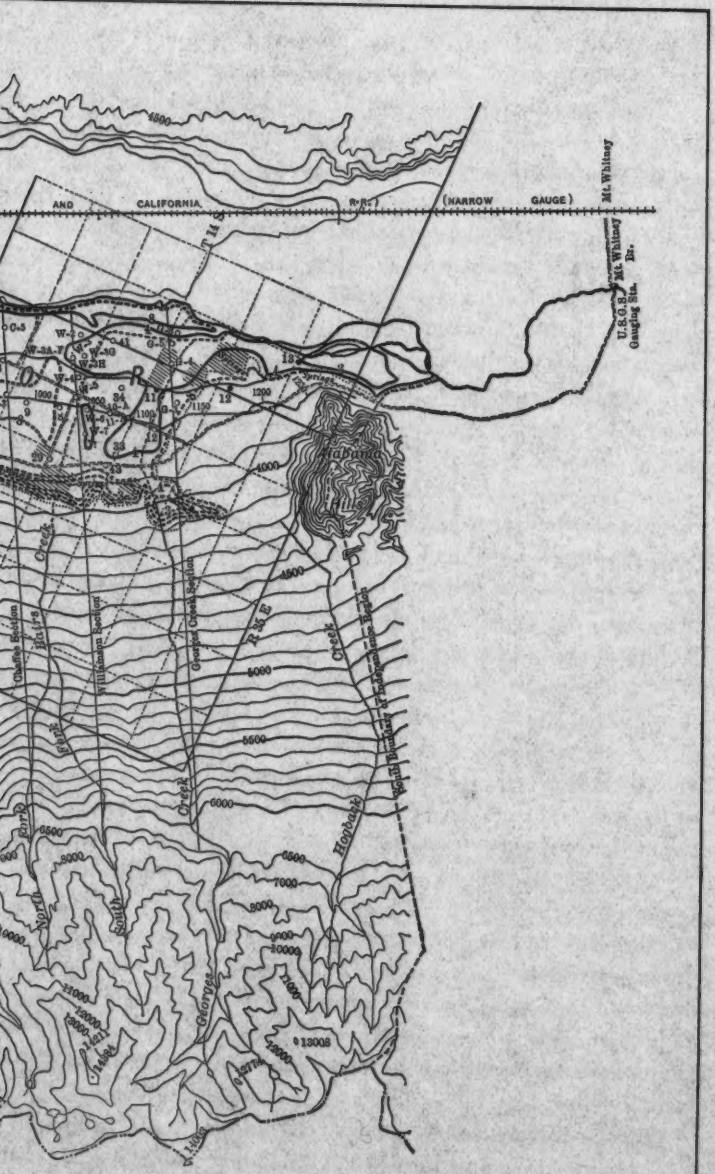
PLATE VI.
PAPERS, AM. SOC. C. E.
APRIL, 1914.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.


AND
CALIFORNIA
R.R.)
(NARROW
GAUGE)
Mt. Whitney
U.S.G.S. Mt. Whitney Gauging Sta.
Mt. Whitney
Owens River
Owens Lake
Owens River Aqueduct
Haiwee Reservoir
Haiwee Pass
Rose Creek
Rose Creek Section
Williamson Section
Owens Section
Olancha Section
Olancha
Cartago
Cartago Section
Hoffmaster
Hoffmaster Section
Keeler
Keeler Section
Alabama Hills
Lone Pine
Lone Pine Creek
Lone Pine Section
Diaz Lake
Diaz Lake Section
Georges
Georges Section
Georges Creek
Georges Creek Section
Alabama
Alabama Section
Alabama Creek
Alabama Creek Section
Independence
Independence Section
Independence Creek
Independence Creek Section
Oak Creek
Oak Creek Section
Oak Creek Canyon
Oak Creek Canyon Section
Fish Springs
Fish Springs Section
Fish Springs Creek
Fish Springs Creek Section
Haiwee
Haiwee Section
Haiwee Creek
Haiwee Creek Section
Haiwee Reservoir
Haiwee Reservoir Section
Haiwee Pass
Haiwee Pass Section
Rose Creek
Rose Creek Section
Owens River
Owens River Section
Owens Lake
Owens Lake Section
Owens River Aqueduct
Owens River Aqueduct Section
Mt. Whitney
Mt. Whitney Section
U.S.G.S. Mt. Whitney Gauging Sta.
U.S.G.S. Mt. Whitney Gauging Sta. Section
Mt. Whitney Section
Owens River Section
Owens Lake Section
Owens River Aqueduct Section
Haiwee Reservoir Section
Haiwee Pass Section
Rose Creek Section
Olancha Section
Cartago Section
Hoffmaster Section
Keeler Section
Alabama Hills Section
Lone Pine Section
Diaz Lake Section
Georges Section
Georges Creek Section
Alabama Section
Alabama Creek Section
Independence Section
Independence Creek Section
Oak Creek Section
Oak Creek Canyon Section
Fish Springs Section
Fish Springs Creek Section
Haiwee Section
Haiwee Creek Section
Haiwee Reservoir Section
Haiwee Pass Section
Rose Creek Section
Owens River Section
Owens Lake Section
Owens River Aqueduct Section
Mt. Whitney Section

(Table 1), and its west boundary practically coincides with the 8-ft. contour.

There is a very striking relation between vegetation and depth to ground-water. On the outwash slopes the vegetation consists of various stunted desert shrubs. In approaching the valley floor at about the 20-ft. contour, sagebrush begins to predominate, and has a luxuriant growth as far east as the 12-ft. contour, where it is replaced by greasewood, rabbit brush, and coarse bunch grass. In the vicinity of the 8-ft. contour, salt grass begins to appear, and farther east, near and within the area inclosed by the 4-ft. contour, it grows luxuriantly. Within the 3-ft. contour, fresh-water grasses thrive where there is sufficient surface water to leach out and carry away most of the alkali, but the salt grass grows well, even where the soil is alkaline. In various portions of the valley floor rabbit brush and greasewood are found where the average depth to ground-water is 4 ft. or more, but grass predominates east of the 8-ft. contour. In areas where the alkali is excessive there is practically no vegetation. In general, grass does not grow where the depth to ground-water exceeds 8 ft., so the 8-ft. contour tends to coincide with the boundaries between meadow and desert lands.

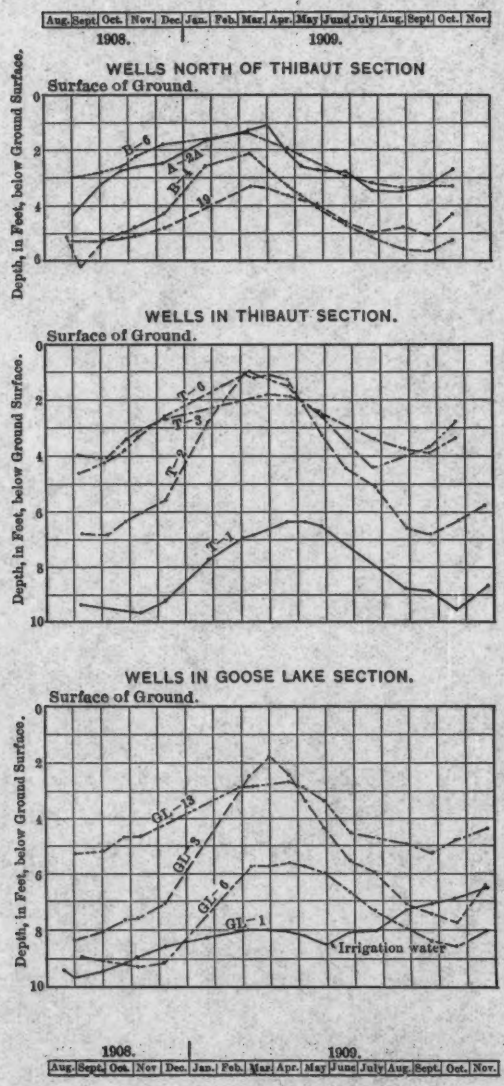
Fluctuation of the Ground-Water Surface.—The surface of the ground-water is continually fluctuating. Both the extent and character of this fluctuation vary widely in different localities and at different times, depending on the proximity to ground-water sources or outlets and the relative rates of ground-water accretion and depletion. Three pronounced types are to be observed in the Independence Basin: (1) broad irregular fluctuations of varying amplitude in the outwash-slope area; (2) slightly irregular periodic fluctuation with wide fixed limits in and near irrigated areas; and (3) a regular periodic fluctuation with comparatively narrow and fixed limits in the valley floor. Special characteristics are also exhibited by wells within certain limited areas, as the result of local ground-water conditions.

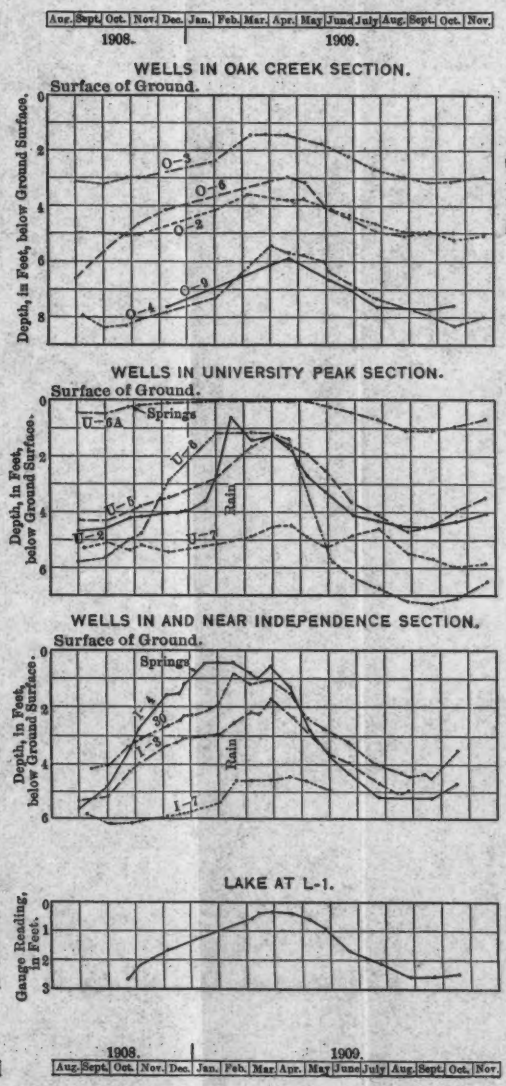
These fluctuations were determined and studied from well observations made by the methods already described. Readings obtained at intervals of from 2 to 4 weeks were sufficient to establish accurately the position of the ground-water surface, as the fluctuations are characterized by great regularity. Most of the wells were observed from August 15th, 1908, to November 15th, 1909, and on 26 of the most

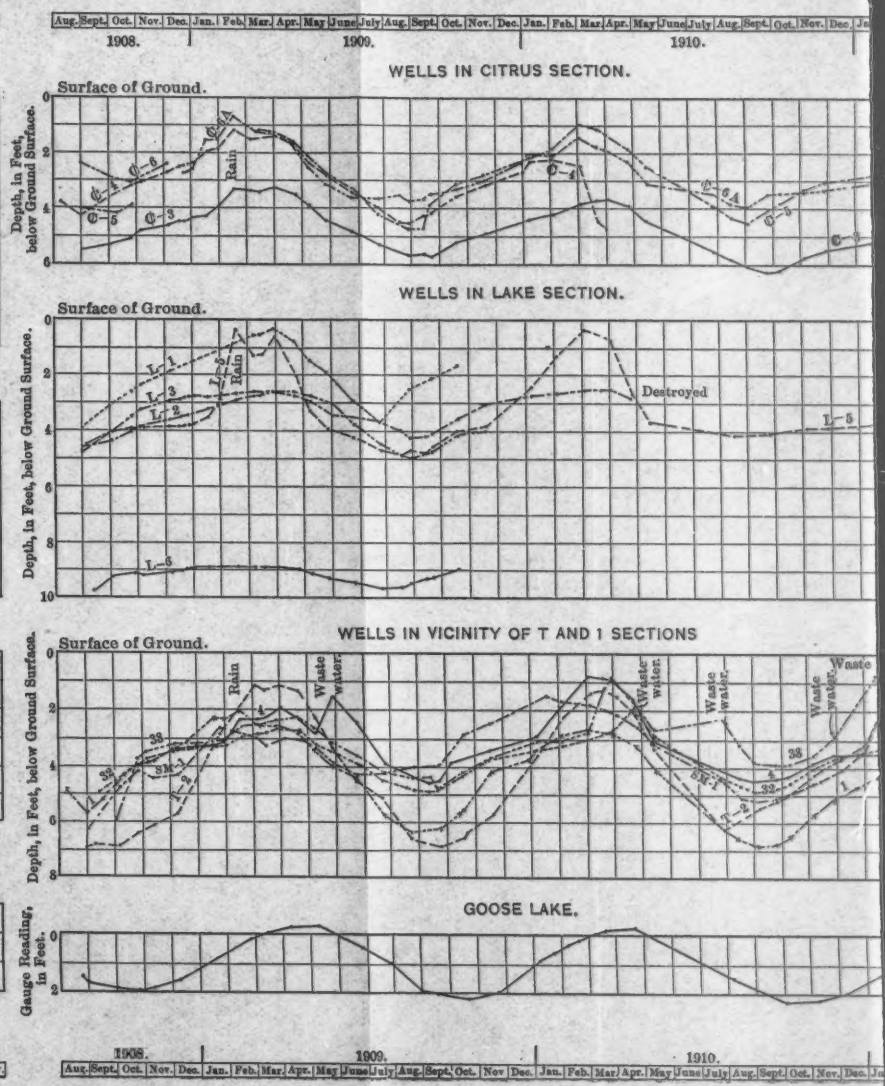
typical wells observations were continued to May 1st, 1911. The fluctuation of the surface of the lake south of Citrus Bridge was observed from August 15th, 1908, to November 15th, 1909, and of Goose Lake from August 15th, 1908, to May 1st, 1911.

The type of fluctuation peculiar to wells on the outwash slope is shown in Plate VII by Wells Nos. 31, 64, 25, 26, and 59, and Citrus No. 1. Water stands 10 ft. or more below the surface in all these wells, the vegetation of the surrounding area is limited to desert shrubs, and there are no alkali deposits on the surface. With knowledge of the sources and movements of ground-water beneath the outwash slopes, the assigned cause for this type would be annual variation in the quantity of water supplied by percolation from precipitation on the intermediate and outwash slopes and from stream channels. This is confirmed by the observations. For example, Well No. 31, which is 7 miles from the base of the Sierra and 500 ft. south of the old channel of Pinyon Creek, exhibits a persistent downward tendency which was partly checked during the summer of 1909 and 1910. The maximum effect of the very wet years, 1906 and 1907, evidently reached this well in 1908 and early in 1909. During the following years the water had a tendency to return to its normal level. This was twice opposed by percolation from the channel of Pinyon Creek, which carried flood-water during a few weeks in June and July, 1909, and for a very short period in 1910. Citrus Well No. 1, which is about $\frac{3}{4}$-mile south of Well No. 31, has similar fluctuations, but in it the maximum effect of seepage from Pinyon Creek is registered 6 weeks later in 1909, and in 1910 is much smaller in quantity. Well No. 64, situated similarly with respect to the mountains, but north of Little Pine Creek, has the same downward tendency, which is checked temporarily during the summer by irrigation in a near-by alfalfa field and a small garden at the well. Well No. 59, which is 2 miles from the base of the Sierra and $\frac{1}{2}$ mile south of Sawmill Creek, had an upward tendency during 1909, due to the percolation from precipitation of the wet winter, 1908-09. In 1910 the water level fell in response to the normal winter of 1909-10. Seepage from Sawmill Creek does not affect this well appreciably. Wells Nos. 25 and 26 exhibit the general tendencies of Well No. 59, but they are in the transition zone between the outwash slope and the valley floor, where there is a periodic back-water effect from the annual rise of ground-water in the grass land.

DIAGRAMS SHOWING FLUCTUATION OF GROUND-WATER SURFACE, TYPICAL OBSERVATIO



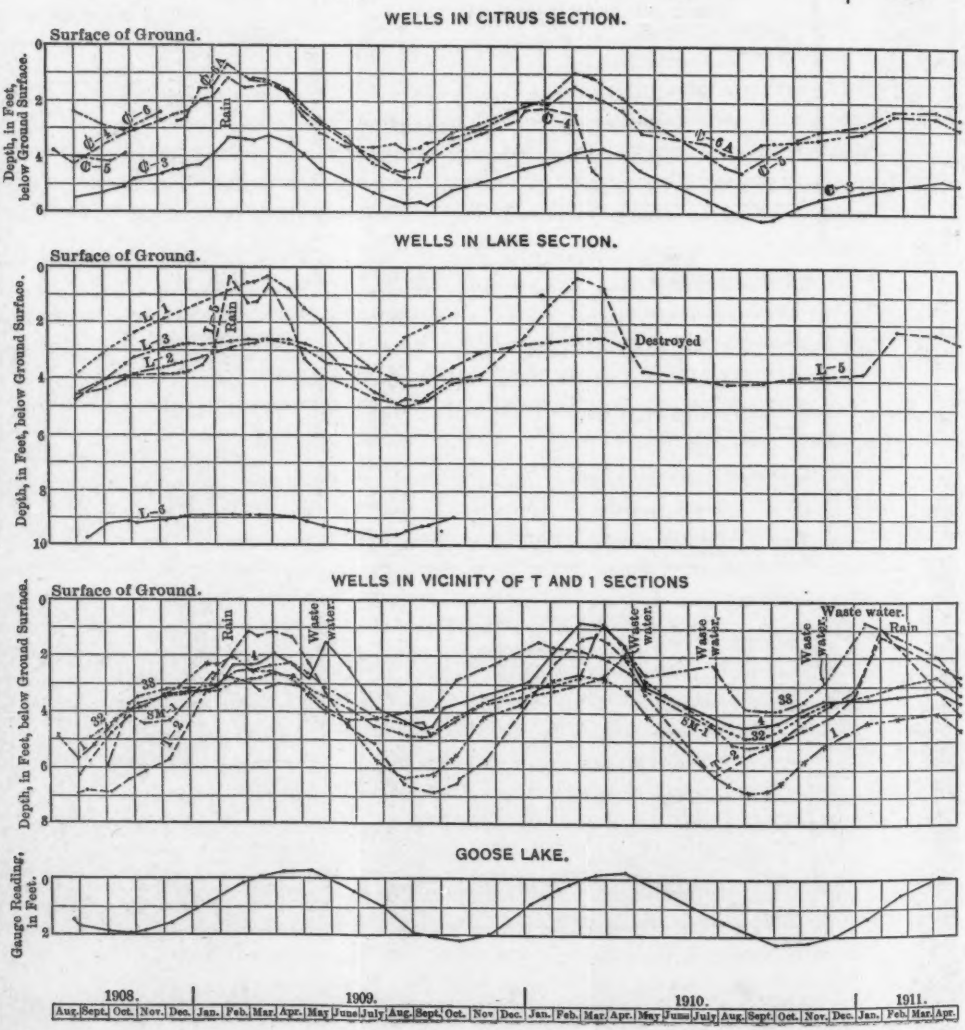




LUCTUATION OF GROUND-WATER SURFACE, TYPICAL OBSERVATION WELLS

Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr.

1908. 1909. 1910. 1911.

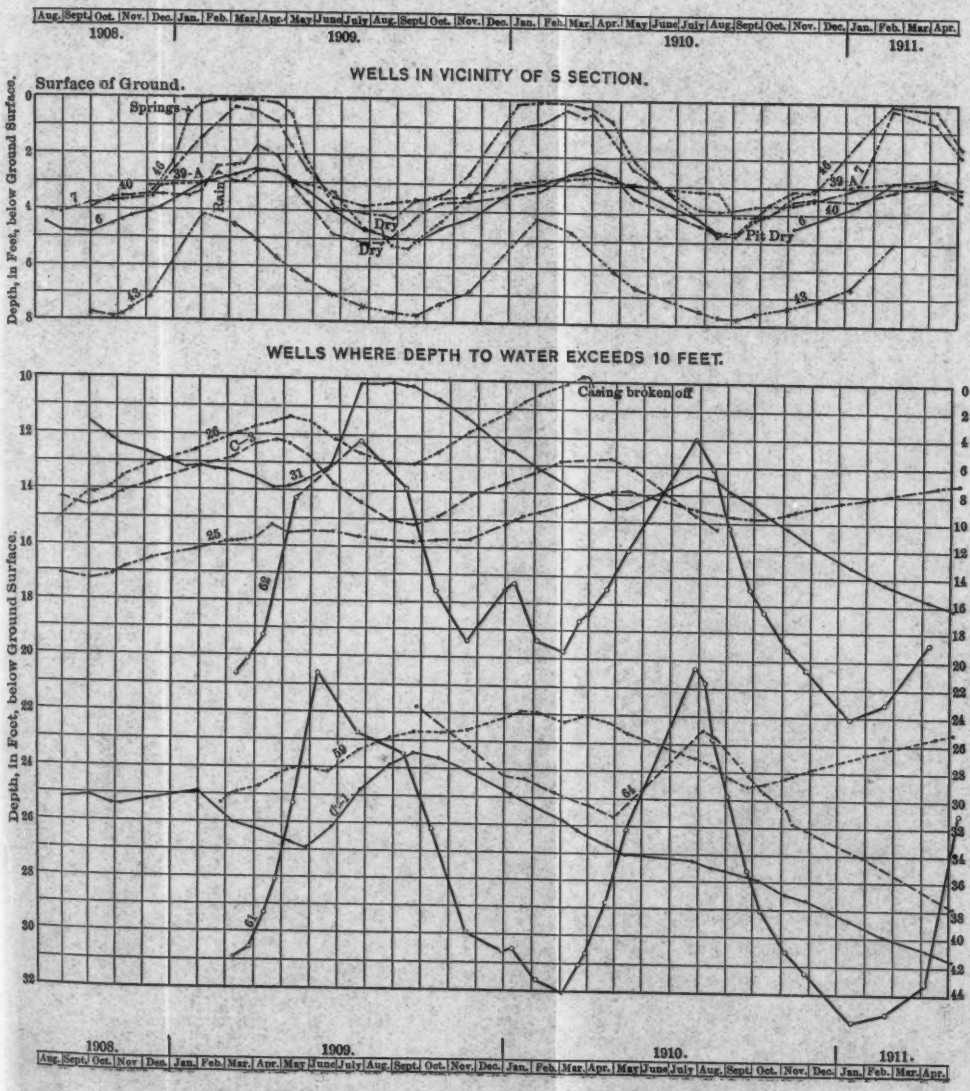


1908. 1909. 1910. 1911.

Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr.

PLATE VII.
PAPERS, AM. SOC. C. E.
APRIL, 1914.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

WELLS IN INDEPENDENCE BASIN



UNIV. OF MICH.

Wells Nos. 61 and 62 illustrate the type of fluctuation characteristic of the irrigated areas of the region. They are in irrigated gardens in the Town of Independence. The form of the curve is periodic, with sharp crests and troughs, the former in July, the latter in January or February. The fluctuation in such wells ranges from 10 to 20 ft. in different portions of the basin. Irregularities superimposed on the broad periodic curve are the result of irregularity in the application of irrigation water.

The fluctuation of the ground-water surface in various parts of the valley floor, other than at the eight wells already mentioned, is shown by 48 typical well records on Plate VII.

Permanent bench-marks were established at each well, and test holes from which measurements to the water surface could easily be made with a steel tape. Before observing, a weight is fastened at the end and the tape is chalked. The end of the tape is then submerged and the difference between the readings at the bench and at the water surface is the depth. Corrections are made in the office when the bench is not at the ground surface. Readings are to feet and hundredths.

Most of these wells are where the average depth to ground-water is less than 8 ft. The adjoining ground surface is more or less crusted with alkali, and, where the alkali is not too concentrated, several species of wild grass grow vigorously. The two lakes and Wells Nos. C-3 and 43 are in areas where the average depth to ground-water exceeds 8 ft., and desert conditions prevail.

The fluctuations observed in all the valley-floor wells are remarkably uniform. When platted, the observations give smooth and regular curves with an annual periodicity. The average time of occurrence of crests for wells in grass or alkali areas is March 28th; the troughs occur on September 20th, 6 months later. Heavy winter precipitation, or the proximity of springs, advances the crests into January or February, but, in the desert areas, the crest lags into April or May. The fluctuation between maximum and minimum levels in normally situated wells ranges from 1.5 to 4 ft. Wells which are near or below springs in the vicinity of intermittently occurring surface water have a greater range, which may reach 7 ft. The average fluctuation for 1908-09, as observed in 122 wells distributed generally over the valley floor, is 3.14 ft. This average represents normal conditions.

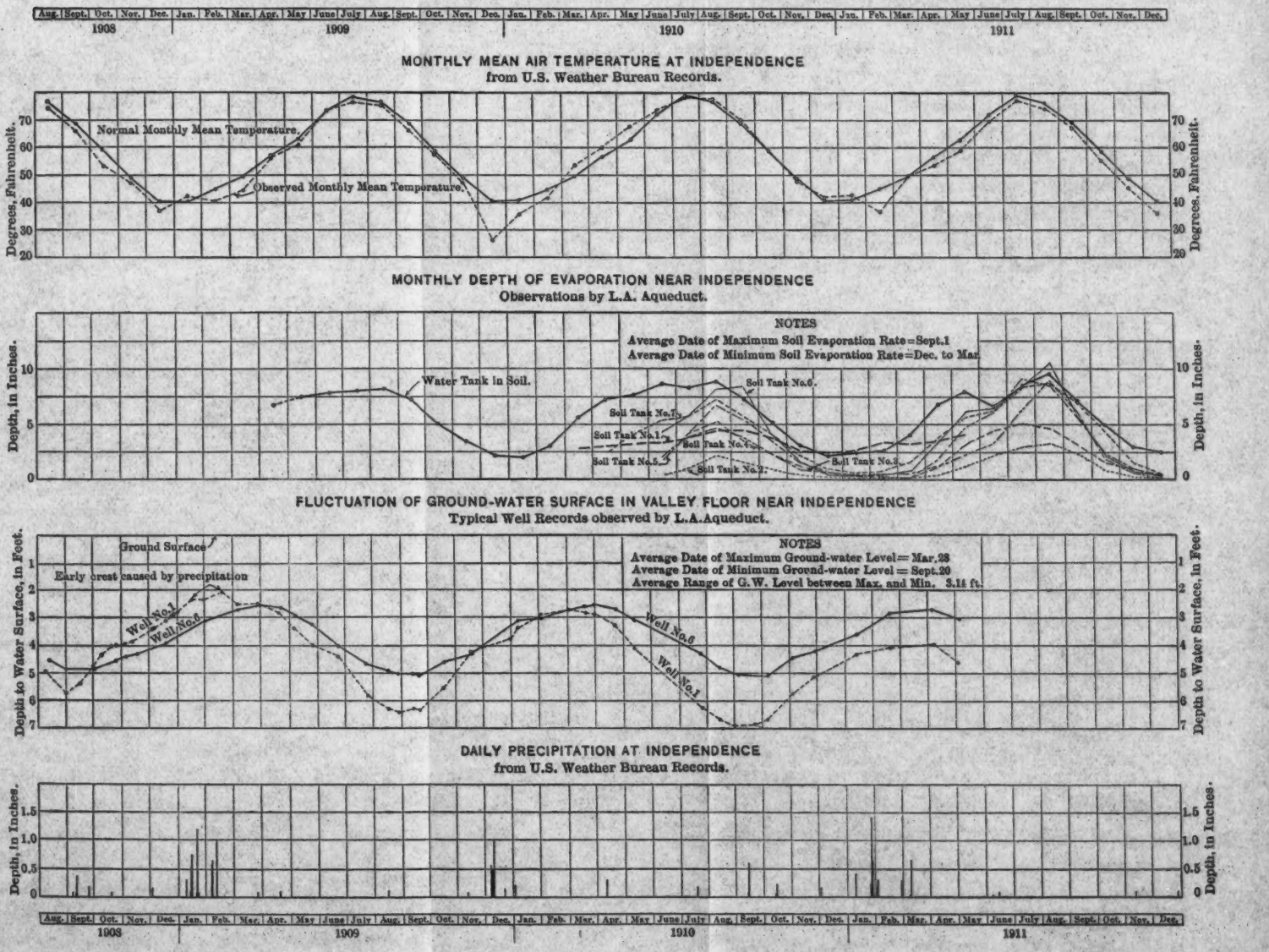
Fluctuation of this type is due to evaporation from the soil and transpiration, processes which are active wherever there is capillary connection between the surface of saturation and the ground surface, or wherever gravity water or capillary water is within reach of plant roots. Two facts have led to this conclusion: (1) the area characterized by capillary connection between ground-water surface and ground surface and by accessibility of ground-water to plant roots is coincident with the area exhibiting this type of periodic fluctuation; and (2) the combined rates of evaporation from soil and transpiration, as observed experimentally, increase and decrease concurrently and in the same ratio with the fall and rise of the ground-water surface.

The first of these facts is indicated by the following observations: Surface incrustations of alkali are now known among investigators to be an indication of evaporation from the soil, and a growth of natural grasses certainly shows the presence of water within reach of plant roots. These manifestations are both strictly confined to valley-floor areas within which the periodic fluctuation is observed. There are valley-floor areas, however, within which the periodic fluctuation occurs, but which have a loose sandy surface devoid of alkali and vegetation. An examination of such areas shows that they are surrounded or bordered by meadow and alkali-crusted land, and further that maximum and minimum ground-water levels exhibit a lag in time of occurrence which varies with the distance from these adjoining lands. (See Plate VII, Well C-3 and Goose Lake.) The fluctuations in these desert areas do not originate within the areas themselves but in the near-by lands, from which they are propagated as annual waves. In general, average depths to ground-water exceed 8 ft. in desert areas but are less than 8 ft. in meadow or alkali lands.

The second fact—that variations of soil evaporation and transpiration are similar to ground-water fluctuations—is indicated by the results of the experiments on evaporation from soil. The maximum rate of soil evaporation occurs about September 1st and the minimum from December 1st to April 1st. The lowest ground-water level occurs about September 20th and the highest level on March 28th (Plate VIII). Thus the critical points in the curves of soil evaporation and ground-water fluctuation are practically coincident as regards time. Furthermore, although the curves are inversely related, their form is remarkably similar. The obvious conclusion is that ground-water fluctuations

PLATE VIII.
PAPERS, AM. SOC. C. E.
APRIL, 1914.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

TEMPERATURE, EVAPORATION FROM SOIL, PRECIPITATION AND FLUCTUATION OF GROUND-WATER SURFACE IN VALLEY FLOOR NEAR INDEPENDENCE.



in non-irrigated portions of the valley fill are the result of evaporation from the soil and transpiration.

Variations from the normal periodic curves occur for three causes: large precipitation, seepage from springs, and seepage from standing or flowing surface water. The infrequency of precipitation sufficient to raise the ground-water surface is shown on Plates VII and VIII. It is practically a negligible factor in ground-water fluctuations. The springs at the upper edge of the outwash slope affect ground-water conditions in their vicinity by stimulating the annual rise and maintaining the ground-water level at a maximum during several months prior to March. (See Plate VII, Wells Nos. 46, U-6A, and I-4.) This results from the decrease in the rate of soil evaporation which allows the accumulation of their discharge in the surrounding soil at a greater rate than in adjoining areas where the rate of supply of underground water is less. Surface water has its source in large springs, waste from irrigation, and the flood waters of mountain streams. It occurs at various times and places, and cannot be considered as a permanent factor in ground-water fluctuations. (See Plate VII, Wells Nos. 4, 38, 39-A, 32, and GL-1.) The irregular fluctuations of ground-water on the outwash slope do not appear in the valley floor because of the relief afforded by the escape of water in springs at the upper edge of the grass land.

Ground-Water Losses.—Ground-water fluctuations within the valley floor consist primarily of the regular annual rise and fall produced by variation in the rate of evaporation. This is indicated by actual observations extending over 3 years, and confirmed by the persistency of various perennial plant species. Hence there must be overflow of ground-water from the valley fill of the region equal to the average inflow by percolation. The possible outlets would seem to be underflow southward through the valley fill, underflow by way of deep fissures, seepage and spring flow into the channel of Owens River, evaporation from spring waters, evaporation from damp soil, and transpiration from vegetation. The first two of these are eliminated by the geology and topography of the region. The slope of the ground-water surface in the valley fill opposite the Alabama Hills does not exceed 8 ft. to the mile, and the material is fine sand and clay, as indicated by the Southern Pacific Company's well at Lone Pine Station. Even if there is a movement of ground-water southward from the

region, it must be exceedingly slow, and it would be entirely intercepted by the alluvial fan of Lone Pine Creek, which has a ground-water surface higher than the valley fill to the north. The granitic formation of the Sierra Nevada and the granite core of the Inyo Mountains are complete barriers against the escape of underground waters through any formation but the valley fill. It has already been shown that there is no seepage flow into Owens River from the water supply of the region. Hence the outlets by evaporation, transpiration, and spring discharge into Owens River are all that remain to be considered.

Soil evaporation and transpiration will be considered, first, for irrigated lands, and second, for the general grass and alkali area of the valley floor. The quantity of water used in irrigating the 3 011 acres under cultivation in the region is about 72 sec-ft. of continuous flow for 6 months (Table 18), which is equivalent to a depth of 8.6 ft. over the whole area. The depth of transpiration from alfalfa during the irrigating season has already been computed as 3.43 ft., or 40% of the total volume used. There is also a small loss through evaporation from the soil during and immediately after irrigations, say 0.85 ft., or 10% of the total. The total loss by evaporation from the soil and by transpiration from irrigated areas, therefore, is 4.3 ft. in depth, or 18 sec-ft., of continuous flow.

The bases for computing the evaporation and transpiration loss from grass and alkali land are the soil-evaporation equations of Fig. 10 and the ground-water contours of Plate VI. The equations were developed for a fixed ground-water surface, but they cover the periods from October 1st to March 31st and from April 1st to September 30th, which practically coincide with the observed periods of rising and falling ground-water. Hence, to cover the natural conditions of fluctuating ground-water surface, average annual depth to ground-water at a given point may be substituted in the equations instead of fixed ground-water depths. The average annual depth to ground-water, in feet, and the depth of evaporation, as determined from the equations for 1911, are given in Table 15 for the non-irrigated areas enclosed by the 3-ft. contour and between the 3- and 4-ft. and 4- and 8-ft. contours. The volume annually evaporating from the whole area enclosed by these contours is equivalent to a continuous flow for the year of 109 sec-ft.

TABLE 15.—Total Evaporation From Grass and Alkali Lands in the Valley Floor.

(Based on 1911 Data.)

Enclosing contours.	Area, in square miles.	Average depth to ground-water, in feet.	Annual Depth of Evaporation, in Inches.			Equivalent flow, in second-feet.
			Summer.	Winter.	Total.	
3 Ft.....	11.89	2.5	36.5	5.2	41.7	36.6
3-4 Ft.....	17.66	3.5	29.6	4.0	33.6	43.7
4-8 Ft.....	25.04	5.5	15.6	0.2	15.8	29.1
Totals.....	54.59	.....	.....	.....	.....	109.4

The water of Blackrock and Hines Springs, and of the small springs along the upper edge of the valley floor, spreads out in many shallow lake basins before reaching Owens River. The loss by evaporation from the surface of these lakes is large. Estimates based on the area of water surface exposed and the evaporation from water in the shallow pan in soil indicate that about 50% of the flow of these springs thus escapes into the atmosphere. As the combined flow is 31 sec-ft., the loss by evaporation from the free water surface is 15 sec-ft. The portion of the remainder which does not flow into Owens River percolates into the soil and escapes by evaporation from the soil and by transpiration.

Two springs derive their waters from percolation and discharge directly into Owens River; these are Upper and Lower Seeley Springs. Their combined average flow is 11 sec-ft. In addition, the Blackrock Springs discharge an average of 7 sec-ft. into the river during November to March, inclusive, which is equivalent to a continuous flow of 3 sec-ft. The total discharge into the river from springs, therefore, is 14 sec-ft.

The grand total ground-water losses, therefore, are 156 annual sec-ft., of which which 127 sec-ft. or 81% is by soil evaporation and transpiration.

Rate of Recharge by Percolation.

From Precipitation.—All portions of the region receive precipitation, but there is wide local variation in the quantities which enter the ground and percolate downward to the surface of saturation. The impervious rock surfaces of the high mountain drainage areas shed

all precipitation which they receive except that lost by evaporation, but accretions to the ground-water from precipitation on the remaining areas of the region are of considerable importance.

Conditions are exceptionally favorable for percolation on the intermediate mountain slopes. As has been stated, the formation is very porous, and practically none of the run-off reaches living streams. All precipitation but that lost by evaporation, therefore, can be considered as percolating downward to the surface of saturation and becoming a permanent addition to the ground-water supply. Snow is practically all melted before May 15th, so that the period of direct exposure to evaporation is not as long as at higher levels, although the rate is greater. Evaporation losses from the moist soil are very small. Before it is melted, the snow blanket protects the soil surface, and by its gradual and uninterrupted melting it fills the capillary spaces to considerable depth, so that gravity water passes downward rapidly. When the snow disappears the rapid drying of the soil surface soon interrupts upward capillary movement, thus preventing further evaporation loss and allowing the percolating water to reach the surface of saturation. In view of these facts, the percolation factor is regarded as being about 0.75 for the more elevated areas receiving approximately 20 in. of precipitation. Less favored areas were assigned smaller factors after a study of their individual characteristics.

The results of computations of the total quantity of percolating water yielded by the intermediate mountain slopes are shown in Table 16. The method was to determine the mean seasonal precipitation at the center of area of each triangular subdivision, multiply this by the area in square miles, and apply a percolation factor. The area of each subdivision and the horizontal and vertical position of its center were obtained from Table 3. Diagrams 7, 8, 10, 11, 13, and 14 on Plate III were used in determining the depth of precipitation. The values differed slightly, as read from the altitude and distance diagrams, and the average was adopted as the most reliable. The total volume of precipitation on the 29.4 sq. miles of intermediate mountain slope is 27 580 acre-ft., of which 19 700 acre-ft. is a permanent addition to the underground water supply of the region. Expressed as a continuous flow, the total percolation from this area is 27 sec-ft.

The outwash slopes yield to the underground supply a much smaller volume of water, which is derived principally from slopes above

TABLE 16.—Percolation From Precipitation Upon Intermediate Mountain Slopes of Independence Region.

(Mean seasonal values.)

Taboose Group of Precipitation Gauges.

Adjoining high mountain drainage area.	Depth of Precipitation on Center of Area, in Inches.*			Volume of precipitation on area, in acre-feet.	Percolation factor.	Amount of Percolation.	
	A.	B.	Average.			Volume, in acre-feet.	Discharge, in second-feet.
Tinemaha	22.2	17.8	20.0	2 320	0.75	1 740	2.4
Red Mountain	23.6	17.8	20.7	2 620	0.75	1 970	2.7
Taboose	22.2	16.5	19.4	4 080	0.75	3 060	4.2
Goodale	18.5	20.0	19.2	2 350	0.75	1 760	2.4
Division	19.9	15.5	17.7	900	0.70	630	0.9
	.....	.....	.....	12 270	.....	9 160	12.6

Oak Group of Precipitation Gauges.

Sawmill	18.2	18.4	18.3	1 290	0.70	900	1.2
Thibaut, North Fork	18.2	18.9	18.6	530	0.70	370	0.5
Thibaut, South Fork	14.5	15.0	14.8	60	0.60	40	0.1
Oak, North Fork	16.4	14.2	15.3	2 950	0.60	1 770	2.4
Oak, South Fork	16.4	15.8	16.1	880	0·70	620	0.9
Little Pine	17.7	15.8	16.8	1 810	0.70	1 270	1.8
Pinyon	18.7	20.8	19.8	3 050	0.75	2 290	3.2
	.....	.....	.....	10 570	.....	7 260	10.1

Bairs Group of Precipitation Gauges.

Symmes	13.4	17.0	15.2	340	0.70	240	0.3
Shepard	12.7	12.4	12.6	650	0.65	420	0.6
Bairs, North Fork	12.4	10.8	11.6	300	0.65	200	0.3
Bairs, South Fork	14.8	12.0	13.4	860	0.70	600	0.8
George	15.8	15.8	15.8	1 760	0.70	1 230	1.7
Hogback	15.2	13.6	14.4	830	0.70	580	0.8
	.....	.....	.....	4 740	.....	3 270	4.5
Grand total	.....	.....	.....	27 580	.....	19 690	27.2

* Depth of precipitation as obtained by the precipitation-altitude diagram is given under A; as obtained by the precipitation-distance diagram under B. The average is taken for use in computations.

the 5 500-ft. contour. Precipitation occurs as snow less often here than on the higher slopes, and usually melts within a few days after falling. The capillary water in the upper layers of the soil thus has opportunity to evaporate after each storm, and it is only when several storms occur in succession that there is enough percolating water to

penetrate the ground beyond possibility of return. The long dry summer and the desert conditions draw all moisture from the ground to considerable depths, and the progress of percolating waters is slow because the capillary spaces must be refilled. Test pits dug in the region of the 4 500-ft. contour, 10 days after a series of storms, showed a penetration of capillary water to a depth of 4 ft. and the entire absence of gravity water. The total precipitation from these storms at this point was about 3.5 in., which, with a 28% available pore space, would represent 1 ft. of completely saturated soil. Considering the evaporation losses, it is not surprising that there was no gravity water within the depth of penetration observed. Observations made at higher elevations after this storm showed gravity water in considerable quantity at a depth of 12 ft. Percolation factors varying from zero to 0.60 were assigned to the several zones of the outwash slope as a result of these field observations.

The results of computations for the total quantity of percolating water yielded by the outwash slopes are shown in Table 17. The whole area of 165 sq. miles was divided into zones lying between contours at 500-ft. intervals from about 4 000 to 6 500 ft., and the zones in turn were divided into groups corresponding with the precipitation gauges. The method of computation was to average the precipitations for adjacent contours obtained from Diagrams 7, 8, 10, 11, 13, and 14 of Plate III. These averages represented the average precipitation for each zone in each group and, when multiplied by the area and the percolation factor, gave the quantity of percolating water which reached the permanent ground-water level. The total annual precipitation on the outwash slopes is 62 000 acre-ft., of which 16%, or 9 800 acre-ft., is effective percolating water. Expressed as a continuous flow, the volume of percolating water amounts to 13.4 sec-ft.

Throughout the valley floor the surface of saturation is so close to the ground surface that capillary connection is maintained during most of the year, and percolation from precipitation is rapid. The depth of penetration is usually slight, however, because precipitation in single storms is small. Several storms in succession or a warm rain on snow will result in a rise of ground-water, but the total average ground-water supply from this source does not exceed 4 sec-ft.

Direct percolation from precipitation, therefore, furnishes a grand total of 44 annual sec-ft. to the underground supply of the basin.

TABLE 17.—Percolation from Precipitation Upon Outwash Slopes of the Independence Region.

(Mean seasonal values.)

Taboose Group of Precipitation Gauges.

Contours bounding precipitation zones.	Area of zones, in square miles.	Depth of Precipitation, in Inches.		Volume of precipitation on zone, in acre-feet.	Percolation factor.	Quantity of Percolation.	
		On contours.	On zone.			Volume, in acre-feet.	Discharge, in second-feet.
Grass–4 500	28.07	6.0, 7.7	6.8	10 180	0.00	0	0
4 500–5 000	8.54	7.7, 9.0	8.4	3 880	0.10	380	0.5
5 000–5 500	8.14	9.0, 10.8	9.9	4 300	0.20	860	1.2
5 500–6 000	5.56	10.8, 12.9	11.8	3 500	0.35	1 220	1.7
6 000–6 500	3.42	12.9, 15.2	14.0	2 550	0.60	1 530	2.1
	53.73	.....	.....	24 360	.....	3 990	5.5

Oak Group of Precipitation Gauges.

Grass–4 500	24.57	4.8, 5.9	5.4	7 080	0	0	0
4 500–5 000	11.78	5.9, 7.3	6.6	4 150	0.05	210	0.3
5 000–5 500	9.23	7.3, 9.1	8.2	4 040	0.15	610	0.8
5 500–6 000	5.82	9.1, 11.3	10.2	3 170	0.30	950	1.3
6 000–6 500	4.82	11.3, 13.6	12.4	3 190	0.50	1 600	2.2
	56.22	.....	.....	21 630	.....	3 370	4.6

Bairs Group of Precipitation Gauges.

Grass–4 500	19.56	3.3, 4.0	3.6	3 760	0	0	0
4 500–5 000	11.68	4.0, 5.2	4.6	2 860	0	0	0
5 000–5 500	9.76	5.2, 6.9	6.0	3 120	0.10	310	0.4
5 500–6 000	9.23	6.9, 8.6	7.8	3 840	0.25	960	1.3
6 000–6 500	5.11	8.6, 10.3	9.4	2 560	0.45	1 150	1.6
	55.34	.....	.....	16 140	.....	2 420	3.3
Grand total	165.29	.....	.....	62 130	.....	9 780	13.4

From Stream Channels.—The most important source of underground water in a desert region is percolation from stream channels. This process is continuous from perennial streams, although it varies with the discharge of the streams and the temperature, as previously indicated. Beneath each stream channel as it crosses the outwash slope is a "ridge" of ground-water rising from the general plane of satura-

tion. The inclination of the slopes of this ridge and the breadth of its base vary periodically with the stage of the creek and the time of year. There is complete saturation within its slopes and a movement of gravity water toward the general ground-water surface. A considerable quantity of water also percolates from intermittent streams.

Percolation from stream channels in the Independence Basin is confined to the creeks draining mountain canyons. There are 17 of of these streams, 11 of which are perennial throughout their channels, 5 are perennial over the upper portion of their channels only, and 1 (Dry Canyon) is entirely an underground stream. The surface flow of the two most northerly of these streams, Tinemaha and Red Mountain Creeks, discharges northward across the Poverty Hills into the Bishop-Big Pine region, but the percolation from their channels is tributary to the Independence Basin. The channels of streams entirely within the basin are continuous from their canyons to the U. S. Geological Survey gauging stations, below which they divide, irrigation ditches carrying all the flow except during the high-water period of wet years, when the excess passes down the natural channels. The problem is thus divided into the determination of percolation above and below the Government gauging stations. The first subject has already been discussed at length and need not be considered here in detail. An inspection of Tables 4 and 5 shows that for the creeks from Taboose to Hogback, inclusive, the total 21-year average discharge at the mouths of the canyons is 130 sec-ft. and at the Government gauging stations 84 sec-ft. If the flow of 2 sec-ft. from Spring No. 2 on Division Creek is included with the canyon discharge, the percolation loss above the Government gauging stations is 48 annual sec-ft. To this should be added 6 sec-ft., as indicated by the diagrams, for Tinemaha and Red Mountain Creeks, making a total of 54 sec-ft.

The quantity lost below these stations is not so easily determined, on account of the numerous channels and irregular flow. Estimates were made on each creek, based on the length of main channel and distributing ditches outside of irrigated areas. The loss per mile was assumed to be the average annual loss per mile for the upper channel of the creek, and the total percolation loss from stream channels below the Government stations was estimated at 25 sec-ft. of continuous flow. This estimate does not include percolation from waste irrigation water or surplus creek water which has passed east of the ranches.

The grand total addition to the underground supply derived by percolation from stream channels, therefore, is 79 annual sec-ft.

From Irrigation.—Irrigation has been practised throughout this region, in connection with farming, for at least 30 years, and is a permanent factor in the underground water problem. The total area under systematic irrigation is approximately 3 000 acres, divided into a number of isolated ranch groups which depend on the mountain creeks for their supply. Oak Creek, the largest of these streams, supplies about 45% of the whole area. The remaining area is divided among eight creeks and the Stevens ditch, which during the period of observation has been largely supplied by the surplus flow of the creeks. The acreage irrigated from each source is given in Table 18. About 50% of this land was originally desert, lying along the lower margin of the outwash slope, and is very porous. The remainder lies in the valley floor, where permanent ground-water is within reach of plant roots and where clay soils predominate. The location of the several areas is shown on Plate VI. Alfalfa and grain are irrigated by

TABLE 18.—ESTIMATED NET VOLUME OF WATER USED FOR IRRIGATION IN THE INDEPENDENCE REGION DURING 1909.

Source of supply.	Area irrigated in acres. *	Duty of water per acre for season, acre-feet. †	Total Volume of Water Used.	
			Acre-feet.	Second-feet for 6 months.
Taboose Creek	(170)	(12)	2 040	5.6
Goodale Creek	(110)	(16)	1 760	4.9
Division Creek	(80)	(16)	1 280	3.5
Sawmill Creek	(90)	(16)	1 440	4.0
Oak Creek, ranch No. 1	109	7.22	790	2.2
Oak Creek, ranch No. 2	49	15.40	758	2.1
Oak Creek, ranch No. 3	155	2.80	435	1.2
Oak Creek, ranch No. 4	260	2.34	609	1.7
Oak Creek, ranch No. 5	38	16.40	623	1.7
Oak Creek	(80)	(16)	1 280	3.5
Oak Creek	(100)	(5)	500	1.4
Oak Creek	(560)	(3)	1 680	4.6
Little Pine Creek	(300)	(14)	4 200	11.6
Symmes Creek	(160)	(5)	800	2.2
Shepard Creek	(280)	(12)	3 360	9.3
George Creek	(160)	(12)	1 920	5.3
Stevens ditch	(310)	(8)	2 480	6.9
	3 011	.....	25 955	71.7

* Areas in parentheses obtained from approximate field observations; other areas obtained by careful field measurement.

† Figures in parentheses assumed from observations on Oak Creek ranches.

flooding, and corn by the furrow method. Three crops of alfalfa are raised each year, and the irrigating season extends from about April 15th to October 15th, although some farmers irrigate 9 months in the year. Grain is irrigated early in the season, and corn late, so that the water is continually used. In most places the use of water is lavish, and no attempt is made to economize it or even to apply the quantity best suited to the crop and soil conditions.

A basis for determining the percolation from irrigation is a knowledge of the duty of water, or the quantity of water used in maturing a given area of crop. This was obtained in 1909 by carefully measuring the quantity of water used daily during the irrigating season on five typical ranches which derived their supply from Oak Creek. On ranches where there was a continual waste from irrigation the surplus water was also measured. Areas in crop were obtained from a careful stadia survey of each ranch.

With conditions on these typical ranches in mind, an examination was made of all other ranches in the region, and values were estimated for the duty of water on each. The number of acres irrigated and in crop was also determined approximately by reference to subdivisions of the public survey. From these data the volume of water used for irrigation was determined, as shown in Table 18. The total volume used during the 6 months, April 15th to October 15th, is about 26 000 acre-ft., equivalent to a continuous flow of 72 sec-ft. during the period. When spread out over 3 010 acres, this represents an average depth of 8.6 ft. This result probably represents an average practice throughout the Owens Valley, for the duty of water measured by the Reclamation Service during the season of 1904 on two typical ranches near Bishop was 7.11 and 9.17 acre-ft. per acre.

The distribution of this water, as regards evaporation, transpiration, and percolation beyond the reach of plant roots, is the next step in computing the ground-water supply from this source. Direct evaporation is relatively small, for the water when spread out over the fields is shaded by the crop and sinks rapidly into the ground. Probably 10% would cover this loss. The transpiration loss from alfalfa during the irrigation season has already been computed as 3.43 ft. depth of equivalent water, or 40% of the average volume of water applied to crops. The transpiration loss from corn and small grains is probably less in this locality, but the direct evaporation loss is greater. There-

fore 50%, or 4.3 ft. depth, represents the quantity of water applied in irrigation in this region which is absorbed by the atmosphere. The other 50% is a permanent addition to the ground-water supply, and is equivalent to a continuous flow of 18 sec-ft. throughout the year.

From Flood Water.—The quantity of percolation from surplus creek water, which spreads out over the valley floor to a greater or less extent, is difficult to determine. Of the 84 sec-ft. average flow at Government gauging stations, 61 sec-ft. are disposed of in channel percolation and irrigation. Possibly 5 sec-ft. of the remainder reach Owens River. This leaves 18 sec-ft. to be divided between evaporation and percolation in the flats between the ranches and the river. The area flooded averages about 5 sq. miles during June and July. The loss by evaporation during this period from a shallow pan in soil was about 24.5 in., and, as the conditions are similar, this represents approximately the loss from shallow flood water. The volume expressed as a continuous flow for two months is 55 sec-ft., or, for a year, 9 sec-ft. The other 9 sec-ft. can be assumed to represent the percolation from this flood water. It is not a permanent addition to the ground-water supply, however, for the surface of saturation is only a few feet below the ground surface in this area, and evaporation from damp soil and transpiration from natural vegetation soon reduce the ground-water surface to its normal position.

Summary of Percolation.—The four sources of ground-water are percolation from direct precipitation, from stream flow, from irrigation, and from flood water in the valley floor.

The first of these yields about 44 annual sec-ft., of which 61% is from the intermediate mountain slopes, 30% from the outwash slopes, and 9% from the valley floor. Percolation from streams yields about 79 annual sec-ft., of which 68% is above Government gauging stations and 32% below. Irrigation yields 18 annual sec-ft. and flood waters in the valley floor 9 annual sec-ft.

The grand total ground-water, therefore, is 150 annual sec-ft., of which probably 75% reaches the deeper strata of the valley fill. The rate of recharge of the basin, as thus determined, differs by less than 4% from the ground-water loss previously computed. The reliability of the data is thus confirmed as well as the correctness of the assumptions.

Safe Yield.

Thus far, this paper has presented conditions as they are found to exist in a natural state. The problems which the engineer has to solve are those connected with the artificial extraction of water from underground reservoirs. First among these is the determination of the safe annual yield or the limit to the quantity of water which can be withdrawn regularly and permanently without dangerous depletion of the storage reserve. A second problem which naturally accompanies the first is the devising of methods for increasing artificially the safe annual yield of reservoirs which are apparently already developed to the limit. The writer will outline his ideas, in the hope that they will suggest a constructive line of discussion which will lead to a better understanding of these subjects.

It is obvious that water permanently extracted from an underground reservoir, by wells or other means, reduces by an equal quantity the volume of water passing from the basin by way of natural channels. This is illustrated by the commonly recognized fact of the drying up of springs and cienagas as the result of heavy pumping. The theoretical limit for safe draft, exclusive of return water, therefore, is the average rate of recharge for a basin. The practical limit, however, depends on the relation of draft to storage capacity, within economic pumping limits. Where the storage capacity is very large as compared with annual draft, the theoretical and practical limits should nearly agree, as the storage reserve can be drawn on in periods of protracted drought. For basins with comparatively small storage capacity, the practical limit will be less than the theoretical. Draft computations may be made with the mass-diagram as ordinarily used in surface storage problems. Storage capacity is determined from the area of water-bearing material, limiting depth for economic pumping, and percentage of voids capable of depletion. The supply is the quantity of water annually absorbed by the porous material of the basin. This may be determined each year by methods similar to those used in the Owens Valley studies.

The draft thus obtained, however, is not the safe yield of the basin, for there are always certain residual losses which cannot be entirely prevented, such for instance as soil evaporation from cienaga lands. These residual losses must be ascertained and deducted from the gross draft. The quantity thus obtained may be persistently with-

drawn from the basin without causing general depression of the water plane to the point where pumping operations must cease for economic reasons.

The determination of residual losses presents difficult problems. Some of the conditions which are responsible for these losses are the following:

1.—The elevation of the impervious rim at the outlet being less than the elevation of the water plane in the lowest depression of the basin, thus allowing ground-water to escape as underflow. The quantity of water thus dissipated depends on the transmission capacity and area of the porous material overlying the rim and the available head. In most cases the volume of water thus lost is relatively very small.

2.—The outlet of springs being at a considerably lower elevation than the general water plane of the basin in the outlet region. Such a condition may exist where an arroya has cut a channel through the impervious rim at a point where the surface falls away rapidly down stream. Such losses are also relatively small.

3.—The occurence of water under Artesian pressure in underlying strata of porous material confined between more impervious layers of fine sand or clay. This is the least recognized, but yet the most important, cause of residual ground-water loss. The effect of Artesian pressure is to force moisture through pores or fractures in the impervious capping and thus maintain a permanent ground-water plane near the surface. The water continually supplied from below is disposed of either by evaporation from the soil surface and vegetation or by escaping at the surface in springs or seepages. These losses persist as long as the Artesian pressure is sufficient to force the water through the overlying strata. The volume of these losses during any period of one or more years bears a functional relation to the average Artesian pressure during the same period. The writer states these conclusions as the result of a careful study of records and conditions in a number of differently situated Artesian basins.

These conclusions are well illustrated by the following facts of common knowledge. First, consider the result of abnormally large precipitation for a period of one or more seasons. The ground-water accretions exceed the losses from the basin, the excess water accumulates in the voids of the porous gravels surrounding the confined strata,

and the free ground-water surface rises throughout the basin. Within the area of confined gravels hydrostatic or Artesian pressure increases. A greater quantity of water is forced through the overlying strata, not only over the area already moist, but from a circumscribing zone within which the pressure was previously insufficient to maintain a shallow ground-water surface. The observed result, therefore, is increased spring and seepage flow and an enlarged area of moist cienaga land from which evaporation occurs. Second, assume a series of dry years. Ground-water storage is depleted, the free ground-water plane falls, Artesian pressure decreases, and less water is forced through the overlying strata. The observed result is decreased spring and seepage flow and shrunken evaporating area. The latter occurs, because, for the new conditions, the rate of evaporation from the outer zone of moist soil exceeds the rate of supply from below. The accumulation of water in the soil is drawn on, lowering the water level to the limit of capillary action. A similar result occurs where relief is afforded to Artesian pressure by the drilling of many deep wells drawing from Artesian strata. In some of the Southern California Artesian basins, which formerly possessed cienaga lands, relief of Artesian pressure by wells and heavy pumping has dried up such lands.

The importance of residual losses, due to Artesian pressure, is forcibly shown by the Owens Valley studies, where it was found that in a natural state 81% of the total yield of the Independence Basin was lost by evaporation from soil and vegetation. Similar conditions, in a slightly less degree, existed in many of the Southern California basins before ground-water development was undertaken. The increased pumping of the last 10 or 15 years has eliminated evaporation losses almost entirely in some of the smaller basins. In the larger basins, such as the San Bernardino Valley and Coastal Plain, the reduction has not been as great, having ranged from 30 to 50% of original evaporation losses in the various basins. In these basins evaporation losses formerly represented from 50 to 75% of the total ground-water supply. Hence, the residual evaporation losses to-day represent from 15 to 35% of the total ground-water supply for the large Southern California basins, and can be said to average 25 per cent.

The quantitative determination of the residual losses from an underground reservoir can only be made after a detailed study of the local

conditions. The factors to be considered are the topography and geology of the basin and its porous filling, the distribution and type of sources of percolating water, the rate of evaporation and transpiration, the depth of capillary action and the character of the soil within the evaporating area, the necessity for irrigation and the value of overlying lands for agricultural crops, the present or probable ultimate method of development of water, and the present or probable future use. The general condition favorable for small residual losses is the possibility of eliminating the evaporating area by lowering the water plane below the reach of capillary action. This may be done by the relief of Artesian pressure, by shallow pumping within the evaporating area, or by drainage. The first of these methods would result in reduced pressures in existing wells in the Artesian area and possibly lowered water levels in the back-water zone, especially if the ground surface has a steep slope. Shallow pumping and drainage, on the other hand, may be physically impractical or prohibitive in cost. Their success depends largely on the existence of shallow water-bearing strata from which water can be readily drawn.

There are three cases which arise in the determination of residual losses: first, a basin already fully developed or suffering from overdraft; second, a basin where the supply is partly developed; third, a basin entirely undeveloped. The first case can be recognized by inspection of the present or former evaporating area in connection with local confirmatory evidence and past records of water levels, yield of wells, etc. The residual losses may be ascertained from observations and measurements of existing conditions. The second and third cases require assumptions as to the method or combination of methods by which residual losses will be reduced to a minimum. These assumptions should be made after a careful study of local physical conditions and the probable future use of the water. The next step is the determination of existing evaporation losses, by contouring the ground surface and water plane, and ascertaining the soil evaporation losses for various depths to ground-water. The final step, namely, the determination of the percentage by which existing losses will be reduced by future development, is as yet largely a matter of judgment. Having arrived at some definite value, however, the residual losses due to evaporation can be computed, and, when combined with the losses from underflow or deep springs, the total quantitative result is obtained.

The thorough investigation of residual losses is essential in any determination of safe yield, and for this reason the writer has discussed the subject in detail.

Passing now from the determination of safe yield as limited by existing conditions to a discussion of methods for increasing safe yield artificially. Obviously, to accomplish the latter, either the rate of recharge of the basin must be increased or the percentage of unused water escaping from the basin must be decreased. Practical methods which suggest themselves are:

(1) Reduction of residual losses to the lowest possible quantity;
(2) Elimination of needless waste of underground water; and
(3) Increased absorption of surface flood waters.

The subject of residual losses has already been discussed at length. The writer wishes to emphasize the fact, however, that a basin has not been fully developed as long as the evaporating area persists in years of drought. The evaporating area is fully as important a criterion of the relation of withdrawals to supply as the water plane. A falling water plane does not of itself indicate overdraft. It is only when a rapid shrinkage and disappearance of the evaporating area accompanies a falling water plane that dangerous overdraft is indicated. Hence, in a closed Artesian basin from which the evaporation area has not disappeared, a greater yield may be obtained, provided an intelligent plan of development is followed.

A very common source of needless waste is from Artesian wells which are allowed to flow when the water is not in use. The wastefulness of this practice is so evident that it need not be discussed in this paper. Its continuance is made possible by lack of recognition of the ultimate effect of the practice among the owners of such wells. The writer feels that it is every engineer's duty and privilege to assist in guiding aright public opinion in matters of common interest, and suggests the subject of conservation of Artesian water as being pertinent in many communities.

The practicability of increasing the ground-water supply by bringing about greater absorption of flood water has been demonstrated in a number of California basins. The most extensive work of this kind is probably that done on the alluvial cone of Santa Ana River in the San Bernardino Valley. The method there used is to divert the flood

water of Santa Ana River in contour ditches from which it is distributed into smaller ditches which in turn subdivide, until finally the water spreads out over the porous alluvial gravels in a thin sheet and is absorbed. During the past few years the volume of water thus stored has averaged 12 000 acre-ft. annually, costing about 15 cents per acre-ft., including interest on the cost of permanent works. The work is capable of further expansion. The ultimate limit will be the ability to handle the violent floods which are of frequent occurrence. The problem is not that alone of controlling the water, but of disposing of silt. The water is normally clear and is free from silt soon after the flood crest passes. Flood water, however, carries great quantities of silt, which deposits as soon as the velocity is checked. This forms an impervious layer of slime which seals the gravels and must be broken up and eventually removed in order to use the same spreading ground continuously.

The conditions on most California streams tributary to closed basins correspond with the Santa Ana. The writer is of the opinion that complete absorption of even ordinary floods on these streams cannot be brought about without temporary surface storage. This must be accomplished either by utilizing storage sites in the stream channel or by construction of contour levees on the alluvial cone. The purpose of such reservoirs would be to act both as settling basins and as temporary storage sites. From these reservoirs the clear water would be released and brought into contact with the absorbent gravels by any method which proved most efficient under the local conditions.

In conclusion it may be said, first, that the rate of recharge of underground reservoirs of the closed-basin type is a definite quantity capable of measurement with a fair degree of accuracy; second, that safe yield is a quantity less than the rate of recharge, its quantity depending on the available storage reserve of the basin and residual ground-water losses; third, that under certain circumstances it is possible to increase the safe yield of a fully developed basin.

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CINDER CONCRETE FLOORS.

By Guy B. Waite, M. Am. Soc. C. E.

To be Presented May 6th, 1914.

Summary.

In the following paper the manner of using reinforced cinder concrete floor slabs by various cities in the eastern part of the United States is described. Especial attention is called to the arbitrary manner of using this material, and the insistence of continuing this arbitrary use, by New York City, as indicated by the latest building codes.

Attention is called to the unit stresses adopted by various cities and to recent data tending to establish unit working stresses from actual tests. The qualifications of this material for the purposes described are discussed, and a comparison with stone concrete is made.

The manner of arbitrary testing is described, and the difference between the conditions of such tests and the actual loadings found in buildings is pointed out.

Analysis is made of the arbitrarily approved reinforced cinder concrete floor slabs, showing the stresses which would be imposed on the materials by the maximum approved loads.

Cinder concrete is described and tabulations are made for the location of the neutral axis and for the carrying capacities, assuming various unit stresses. The influence of the stresses due to an excess of steel or to an excess of concrete is discussed.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Analyses of stresses allowed by such tests indicate that in some cases approvals have been given for dangerous construction; it is pointed out that arbitrary tests can be made which indicate carrying capacities not possible under the conditions actually found in buildings; and it is shown by formulations and tabulations that reinforced cinder concrete may be easily and uniformly designed by adopting working unit stresses, and that several cities have adopted such stresses for this material.

The writer believes that in this paper he has amply demonstrated that the only manner in which this material can ever be uniformly used under various conditions is for each locality to adopt unit stresses suited to its own conditions.

Where cinders of good quality can be obtained, cinder concrete, reinforced with steel, for fire-proof floor construction, is used very extensively in the eastern part of the United States. It is estimated that more than 50% of the large fire-proof constructions of Philadelphia, Pa., have reinforced cinder concrete floor slabs, and New York City has probably a larger proportion.

Floor slabs of stone and gravel concrete, reinforced with steel, of the same general form as cinder concrete, have been systematized by engineers so that they are being used on a rational basis; but cinder concrete has been allowed to straggle along, without much attention from engineers in general, and is being used in a varied manner. Most large cities, such as Philadelphia, Pa., Boston, Mass., and Chicago, Ill., require cinder concrete for floor slabs to be designed, in thickness and in quantity of reinforcement, with certain values of unit stresses as a limit.

In New York City, and in most small cities of the eastern part of the United States, unit stresses are not considered in the use of such concrete. In these places, the use is either entirely arbitrary or a separate test is required for each condition.

In New York City, since the beginning of the use of cinder concrete floors, in 1896, there have been some 200 approvals for reinforced floor slabs of this material, based on load tests. These approvals were for varying spans, thicknesses of concrete, and quantities of reinforcement. The tests have generally been conducted with a uniformly

distributed load—of pig iron, bags of cement, or sand—imposed on the slab.

As will be shown in detail, the results of former tests, which include all principal constructions in use, are extremely variable. The writer has been informed that since 1911 concentrated load tests are required for all new approvals, by the Borough of Manhattan, New York City. The result is that now there are approvals of concrete floor slabs in which the effective depth is 3 in. (4 in. total thickness) and the reinforcement is less than ½ lb. per sq. ft. on an 8-ft. span, and with a live floor load of 250 lb. per sq. ft. In these slabs the concrete is in the proportion of 1:2:5.

One of the dealers in reinforcing materials for cinder concrete publishes a statement of what he calls its "authorized use" by the Boroughs of Greater New York City. This is a 1:2:4 mixture, with a 5-in. effective depth up to 12-ft. spans for live loads of 60 lb. per sq. ft.—or a total load of about 120 lb.

Each contractor in New York City is held strictly to the details of construction for his approvals. For instance, if the approval allows a live load of 250 lb. per sq. ft. up to 6-ft. spans, and it is desired to use the construction for a live load of only 60 lb. per sq. ft., on 6 ft. 6-in. spans, this construction will not be allowed for this excess of 6 in. in span, though stressed less than with the approved loads.

Moreover, stone and gravel reinforced concrete floor slabs are required to be designed according to the following values: Extreme fiber stress in concrete, 650 lb. per sq. in.; maximum tension in mild steel, 16 000 lb. per sq. in.; ratio of moduli, 15. According to these values, stone concrete slabs in some instances would be designed with a greater thickness and have greater reinforcement than the approvals for cinder concrete.

The writer believes that the excuse, that cinder concrete is so extremely variable as not to permit of the assignment of unit stresses, is not well founded. The fact that cinder concrete is being used extensively in engineering construction is sufficient reason for assigning, for safety, some unit stresses, in order to insure uniformity. The present manner of making arbitrary approvals has led to confusion, various forms of reinforcement obtaining approvals in which the spans, thicknesses of concrete, quantities of reinforcement, and floor loads vary beyond description.

The proposed building codes of 1912 and 1913 have persistently continued the present manner of using cinder concrete (with slight modifications) for floor constructions.* In these codes, the manner of designing and constructing cinder concrete floor slabs is arbitrarily specified. In the proposed code of 1913, the allowed carrying capacity of the slab with 3 in. effective depth on 8-ft. spans is increased (over the 1912 code) to a live load of 250 lb. per sq. ft.

It is on account of these conditions in the uses of cinder concrete that the writer has been prompted to bring this matter before the Society, with the hope of interesting the Engineering Profession in a subject which should be of vital concern to many.

Cinders.—Cinders for concrete floor construction are understood to mean either hard or soft coal ashes coming from boiler plants. Coal ashes from a stove or a small house boiler are generally too finely burned to be of use in this kind of concrete. Cinders from either hard or soft coal, and having sharp particles greatly in excess of the smooth ashes, make a concrete of considerable strength and of good fire-resisting qualities.

Cinders vary in size from the consistency of coarse building sand to clinkers several inches in diameter. Some hard-coal cinders run so perfectly in grade that good concrete is made without the addition of sand; more often, however, sand must be added. The quantity of sand required to fill the voids is variable, but the usual specification is 2 parts of sand to 5 parts of cinders.

After a long experience in the use of cinder concrete, the writer believes that there should be no danger in assigning working stresses to the material, these stresses being taken from tests on known mixtures in which cinders of the lowest grade permitted are used. If the material is too poor to have such working stresses assigned, it seems evident that cinder concrete is not a safe material for such an important part of the structure as the floor slab.

Cinder Concrete as Fire-Proofing.—Since 1896 some 82 different floor constructions have been tested in the United States by fire and water. Nearly all these tests were made under the auspices of the Bureau of Buildings of New York City, on full-sized floor slabs. About 40 of the tests were on reinforced cinder concrete floor slabs, some 23

* Sec. 113, proposed code of 1912; and Sec. 104, proposed code of 1913.

on stone or gravel concrete floor slabs, 10 on some form of hollow tile, and the others on special floor constructions.

The test structures were about 14 ft. square and 9 ft. high, with a fire grate at the bottom and the floor slabs to be tested forming the ceiling. The average temperature on the tested floor slabs was 1 700° Fahr. maintained for 4 hours. Following this there was a water test of 60 lb. pressure through a 1¼-in. nozzle applied for 10 min. to the under side of the slabs. During the fire test a load of 150 lb. per sq. ft. remained on the entire surface of the slabs. After the fire test the slabs were subjected to a total load of 600 lb. per. sq. ft. Fig. 1 is a view inside one of these test structures after a fire test.

In these tests, the cinder concrete floor constructions withstood the fire and water better than any other material. In the writer's opinion, the reason for this is not due to cinder concrete being more fire-proof than the others, but simply to the fact that it is not ruptured and destroyed by the expansions and contractions caused by fire and water.

Hollow tile, as a material, may be more fire-proof than cinder concrete, but the stresses due to the expansion of the exposed surface, destroy it.

The test structure, an inside view of which is shown, was built entirely of cinder concrete, mixed 1 to 4 without sand, and withstood four different fires before being torn down to make room for improvements. Only the surface, for a depth of less than 1 in., was affected; this was dehydrated, but the burned cinder concrete was still intact, and protected the remainder of the material. In this concrete there was at least 5% of unburned pea coal which was unaffected by the fire.

Arbitrary Testing of Cinder Concrete.—In New York City after each type of cinder concrete construction had qualified for fire-proofing, the variations in spans, carrying capacity, etc., were determined by each manufacturer by constructing the slabs he proposed to use and testing them with uniformly distributed loads under the supervision of the Bureau of Buildings.

This was the method used for the approvals of the constructions shown, with the exception of Fig. 5, which was constructed for fire test. These constructions were taken from the files of the Bureau of Buildings from among some 200 similar approvals.

For comparison with the approved cinder concrete floor slabs shown by Figs. 3 to 7, the writer gives the relative thicknesses and the quantity of reinforcement for slabs under similar conditions according to the unit stresses used by Philadelphia, and also the thicknesses and quantity of reinforcement required by New York City for 1:2:4 stone concrete.

It is to be noted, in comparing the stone or gravel concrete construction (Figs. 11 to 13) with the cinder concrete, that the former is made unquestionably continuous, and the latter, in most cases, is cut into and is simply supported by the steel beams (Figs. 3 to 10, and 14 and 15).

The approvals for the longest spans with the highest carrying capacity were based on tests with slabs which were continuous over supports (Figs. 3 to 7). In practice, it is seldom that cinder concrete floor slabs between steel beams can be made continuous throughout a floor.

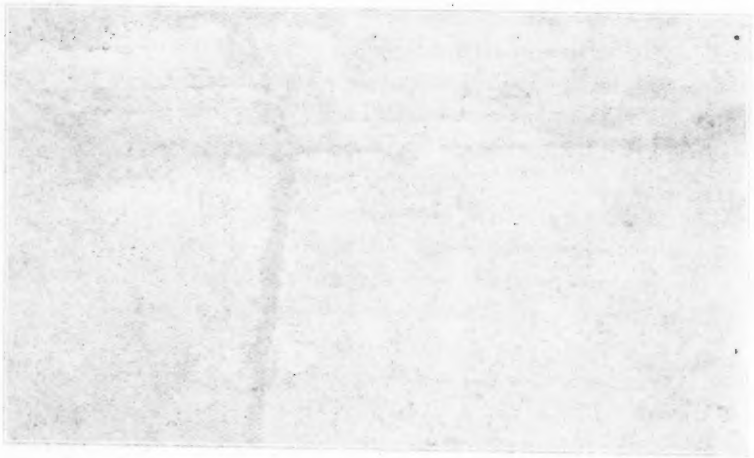
In Figs. 14 and 15, which are assumptions for illustration, it will be noticed that the reinforcement is simply laid over the top flanges of the supporting steel beams. In practice, there is no way of taking up the slack in the reinforcement coming over the tops of the beams, and the consequences are that under such conditions there can be no continuous action—such as engineers understand—in the combined steel and concrete structure. Unless considerable slack is allowed on the sides of each supporting beam, the tamping of the concrete works the reinforcement to the top of the slab.

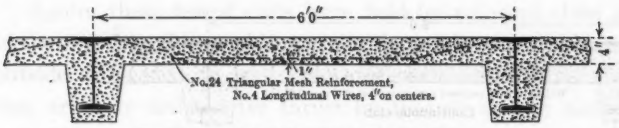
In buildings where a wooden floor is used, the sleepers are generally laid directly on the steel beams, and consequently the concrete is kept below the top flanges of the beams. Here, therefore, the concrete is not continuous in any part of the floor; but in the interior spaces on all floors, whether continuous or not, the channels on the outside spans, and against openings in floors, cannot have either concrete or reinforcement continuous over them (Figs. 14 and 15).

To any one acquainted with the actual construction of cinder concrete floors, it must be apparent that some parts of each floor will not admit of being constructed with the same care that might be attained in a test construction; therefore it does not follow that because a slab was continuous in the test, it is so in actual practice.

FIG. 1.—CEILING AND SIDE OF CINDER CONCRETE, AFTER THE SECOND FIRE. PLASTER ON PART OF CEILING.

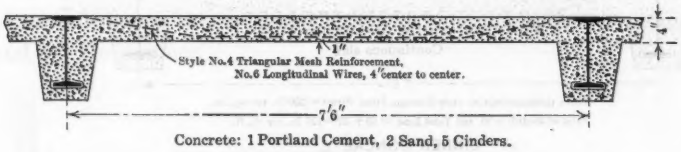
FIG. 2.—ARBITRARY TEST LOAD. SAND IN BAGS ON 4-IN. SLAB, 36 IN. WIDE AND 6 FT. LONG.





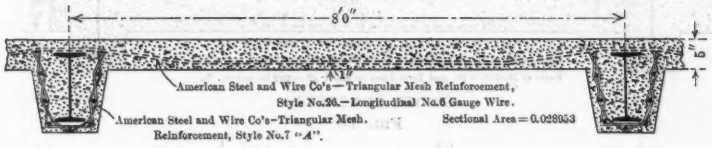
Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.
Working Load: 400 lb. per Sq. Ft.

FIG. 3.



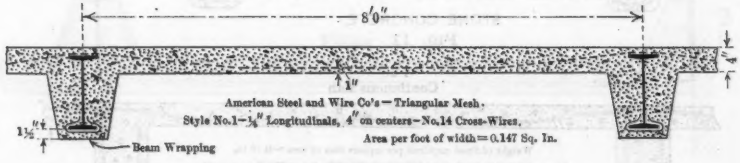
Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.
Working Load: 95 lb. per Sq. Ft.

FIG. 4.



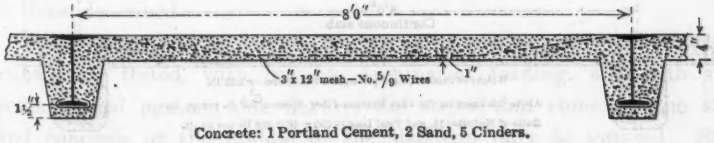
Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.
Working Load: 150 lb. per Sq. Ft.

FIG. 5.



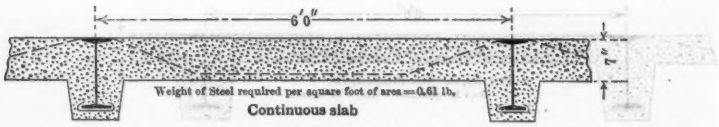
Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.
Working Load: 200 lb. per Sq. Ft.

FIG. 6.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.
Working Load: 250 lb. per Sq. Ft.

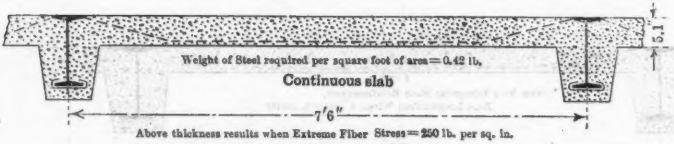
FIG. 7.



Above thickness results when Extreme Fiber Stress = 250 lb. per sq. in.
Ratio of Modull = 30, and Total Load = 400 + 32 = 432 lb. per sq. ft.

CINDER CONCRETE.

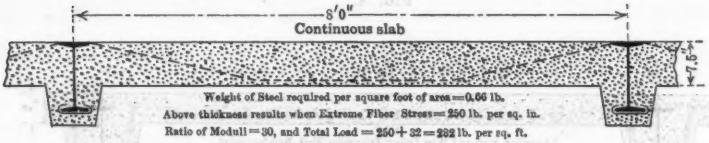
FIG. 8.



Above thickness results when Extreme Fiber Stress = 250 lb. per sq. in.
Ratio of Modull = 30, and Total Load = 95 + 32 = 127 lb. per sq. ft.

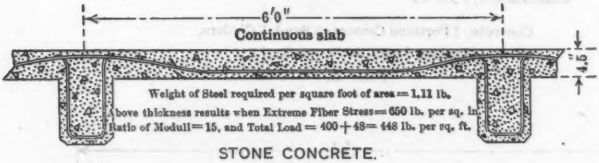
CINDER CONCRETE.

FIG. 9.



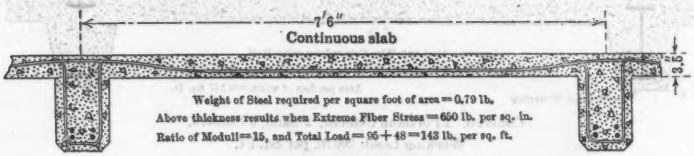
CINDER CONCRETE.

FIG. 10.



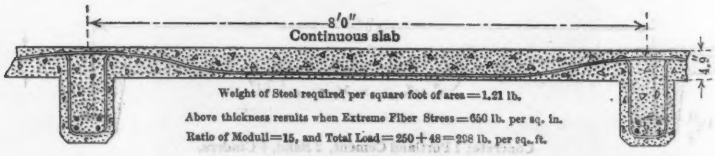
STONE CONCRETE.

FIG. 11.



STONE CONCRETE

FIG. 12.



STONE CONCRETE.

FIG. 13.

Again, these tested slabs were held by adjacent slabs on each side of the supports, thus enabling each side to take thrust. Now, in practice, outside slabs next to walls, elevator shafts, stairways, show windows, etc., are only resisted for thrust by tie-rods and by concrete slabs on one side.

If the writer is correct in his statement that outside slabs, etc., are not under the same conditions as the tested ones, where the reinforcement and the concrete is carefully made continuous, then this form of so-called continuous construction is being used in an unapproved and unsafe manner.

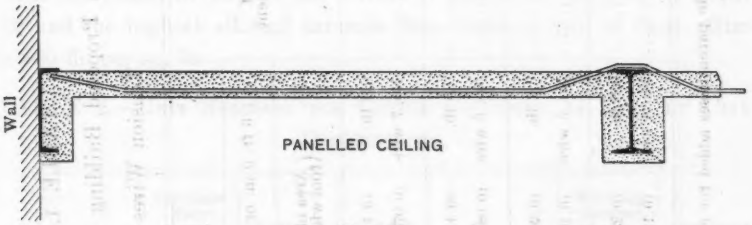


FIG. 14.

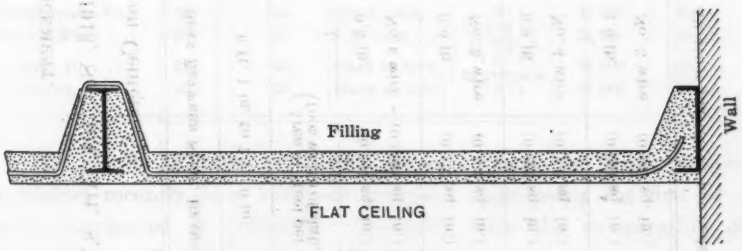


FIG. 15.

The tests on which the approvals were based had the kind of loadings heretofore mentioned. Fig. 2 shows a test loading similar to those described.

With the conditions of continuous construction under which these slabs were tested, with possible arching of loading, and with supports braced against side flexure, the stress that came on the steel and concrete at the middle of the slab can only be guessed. Still, according to the latest proposed building code (that of 1913) tests and

TABLE 1.—REINFORCEMENT.

(From Proposed Building Code of 1913, Sec. 104, Art. 8.)

Tension Wires 4 Inches on Centers.

Live load per square foot of floor area.	Type of reinforcement.	Span Between Steel Beams.					
		6 ft. 0 in. or less.		6 ft. 1 in. to 7 ft. 0 in.		7 ft. 1 in. to 8 ft. 0 in.	
		(Area of steel per foot width of slab.)		(Area of steel per foot width of slab.)		(Area of steel per foot width of slab.)	
100 lb. and less	Bars	0.4 lb.	(0.118 sq. in.)	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)
	Wire mesh	No. 7 wire	(0.0735 sq. in.)	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)
101 to 150 lb.	Bars	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)	0.8 lb.	(0.236 sq. in.)
	Wire mesh	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)
151 to 200 lb.	Bars	0.7 lb.	(0.206 sq. in.)	0.9 lb.	(0.265 sq. in.)	1.1 lb.	(0.324 sq. in.)
	Wire mesh	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)
201 to 250 lb.	Bars	0.8 lb.	(0.236 sq. in.)	1.0 lb.	(0.294 sq. in.)	1.2 lb.	(0.353 sq. in.)
	Wire mesh	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)	No. 2 wire	(0.162 sq. in.)

Matter in parentheses added for comparison. All other matter as per code.

approvals of that kind were to have preference over unit stresses. The following is quoted from Sec. 104, Art. 8, of this proposed code:

"The concrete is made 1, 2, 5; slab thickness, 4 in., with reinforcement 1 in. from bottom.

"Reinforcement must be as per tabulations."

It will be noticed from Table 1 that when the reinforcement is of wire mesh the area required is only about one-half as great as for any other form of reinforcement—even with the same kind of wire but without the mesh.

Use of Cinder Concrete by Cities Other than New York.—The cities mentioned in Table 2 use a ratio of moduli of elasticity of about 30, and the highest allowed extreme fiber stress in any of these cities is 300 lb. per sq. in.

TABLE 2.—UNIT STRESSES FOR CINDER CONCRETE, AS USED BY CITY DEPARTMENTS.

City.	Extreme fiber stress, in pounds per square inch.	Ratio of moduli of elasticity.	Kind of cinders used.	Mixture.	Working stresses in steel, in pounds per square inch.	Preferred reinforcement.
Philadelphia, Pa.	250	30	Hard coal...	1 : 2 : 4	16 000	None.
Boston, Mass....	300	25	Soft coal....	1 : 2 : 4 to 1 : 2½ : 5	16 000	None.
Chicago, Ill.....	245	30	Hard or soft.	1 to 8 aggregate.	18 000	None.
Baltimore, Md...	300	30	Hard or soft.	1 : 2 : 4.	15 000	None.

The results of tests on cinder concrete quoted in Table 3 were published recently in a technical journal,* these being the first of a series conducted at Columbia University with the co-operation of the Bureau of Buildings of New York City.

Though the limited tests in Table 3 would seem to show that a ratio of moduli of less than 30 could be used, the writer has been informed that specimens "C" were from extra good concrete, and that "A" and "B" are nearer the average material. In this case the ratios used by Philadelphia, Chicago, and Baltimore seem to be about right. As will be seen later, a less ratio of moduli will give a less carrying capacity, other things remaining the same.

**Engineering News*, October 9th, 1913.

In the tests at Columbia University, just referred to, it is the feeling, at the present stage of these tests, that an extreme fiber stress of from 200 to 250 lb. per sq. in. on cinder concrete is all that can be used conservatively.

Only the tests up to 6 months old have been made; the final tests may alter this assumption, but, for discussion, these values are sufficiently correct.

TABLE 3.

"WEIGHT AND COMPRESSIVE STRENGTH OF CINDER CONCRETE AS USED IN FIREPROOF FLOORS, NEW YORK CITY.

(The table covers 120 samples. Each figure given is the average of ten samples.)

Zone.	A.	B.	B2.	C.
Mix	1:2:5	1:1:5	1:2:5	1:2:5
Weight, lb. per cu. ft.	107	100	107	109
One-month Test:				
Crushing Strength, lb. per sq. in.	407	507	818	980
Mod. of Elast., lb. per sq. in.	924 600	857 400	1 280 000	1 492 000
Two-month Test:				
Crushing Strength, lb. per sq. in.	701	662	1 254	1 035
Mod. of Elast., lb. per sq. in.	1 134 000	1 080 000	1 740 000	1 428 250
Six-month test:				
Crushing Strength, lb. per sq. in.	933	754	1 744	1 478
Mod. of Elast., lb. per sq. in.	971 000	1 050 000	1 848 000	1 276 000

NOTE.—B was hand-mixed; A, B2, and C were machine-mixed. The modulus of elasticity was determined at a point of the elastic curve corresponding to one-fourth the ultimate strength."

Designing Reinforced Concrete Floor Slabs.—Solid slabs of cinder concrete are not weak in shear, so that the discussion of vertical and diagonal shear will not be necessary. In the writer's observation of many breaking tests on cinder concrete floor slabs, there has never been a failure near the supports; almost uniformly, the breaking has taken place near the center of the slab. Usually, the top of the central portion has spalled off (cup fashion), apparently due to horizontal shear and compression, after which the slab suddenly collapsed.

Adhesion.—The adhesion of concrete to various kinds of reinforcement is shown by Table 4, the results of a series of tests on 1:2:4 cinder concrete, made in 1904 by Messrs. H. B. Gaylord and H. A. Pratt, at Stevens Institute of Technology. The cement was Lehigh Portland, and the cinders were as found coming from a boiler plant. The concrete blocks were 6 by 6 in., and the steel was embedded in the center.

It may be noted from Table 4 that the plain bars held in adhesion quite as well as the deformed bars; the average adhesion of surface embedded being about 200 lb. per sq. in. This same fact, as to the relative adhesion of plain and deformed steel in stone concrete, has already been shown in numerous tests. Therefore, it will be considered that the form of the reinforcement is immaterial so long as it has sufficient surface embedded.

Bending.—Proceeding on the ordinary assumptions of the common theory of flexure, Fig. 16 is a stress diagram, at the right of which

TABLE 4.—RESULTS OF ADHESION TESTS ON 1:2:4 CINDER CONCRETE.

No. of specimen.	Length embedded, in inches.	Perimeter, in inches.	Area of cross-section, in square inches.	Shape of bar.	Kind of concrete.	Pull, in pounds per inch.	Pull, per inch of perimeter.	Pull per square inch.
1	6¼	1.75	0.094	De Mann.	Cinder, 1:2:4: Stone.	396	1 414	206.2
2	11⅜	1.75	0.094	"	"	598	3 885	247.2
3	6¼	1.75	0.094	Flat.	"	376	1 343	214.6
4	16	1.75	0.094	"	"	250	2 285	142.8
5	18	1.75	0.094	"	"	230	2 371	131.4
6	23	1.75	0.094	"	"	176	2 072	89.8
7	5	1.57	0.196	Round.	"	400	2 730	255.0
8	6⅛	1.57	0.196	"	"	553	2 076	339.2
9	9	1.57	0.196	"	"	647	3 710	412.2
10	13	1.57	0.196	"	"	379	5 460	196.0
11	16	1.57	0.196	"	"	417	4 331	277.0
12	28¾	1.57	0.196	"	"	239	4 188	145.0
13	6	1.625	0.156	½ by ⅛-in.	"	500	1 834	307.6
14	10¼	1.625	0.156	"	"	146	914	90.0
15	12	1.625	0.156	"	"	500	3 668	308.0
16	17¼	1.625	0.156	"	"	345	3 661	212.3
17	19¾	1.625	0.156	"	"	310	3 778	191.0
18	27¼	1.625	0.156	"	"	232	3 895	143.0
19	4¾	3.25	0.625	1 by ⅝-in.	"	990	1 477	311.0
20	6¾	3.25	0.625	"	"	800	1 661	246.3
21	14¾	3.25	0.625	"	"	665	3 015	204.6
22	20¼	3.25	0.625	"	"	680	4 353	215.0
23	26¾	3.25	0.625	"	"	645	5 303	193.0
24	11½	3.125	0.203	1 by ¾-in. channel	Stone.	895	3 168	247.9
25	18	3.125	0.203	"	"	610	3 505	194.7
26	22½	3.125	0.203	"	"	523	3 433	168.8
27	6⅜	3.125	0.203	"	Cinder.	840	1 715	273.1
28	8⅛	3.125	0.203	"	"	848	2 304	271.1
29	16¾	3.125	0.203	"	"	245	1 120	166.8
30	17	3.125	0.203	"	"	555	3 019	177.6
31	20¼	3.125	0.203	"	"	435	2 216	141.0
32	22½	3.125	0.203	"	"	476	3 435	154.0
33	35⅝	3.125	0.203	"	"	294	3 357	93.9
34	6½	1.00	0.0625	Ransome.	"	431	2 800	439.8
35	7½	1.00	0.0625	"	"	377	2 825	376.6
36	10	1.00	0.0625	"	"	409	4 085	408.5
37	13¼	1.00	0.0625	"	"	334	4 435	334.8
38	22	1.00	0.0625	"	"	270	5 250	238.6
39	20½	1.00	0.0625	"	"	755	2 000	75.4
40	6	1.00	0.0625	Square.	"	442	2 650	441.6
41	7¾	1.00	0.0625	"	"	322	2 500	332.5
42	11½	1.00	0.0625	"	"	322	2 500	217.4
43	15½	1.00	0.0625	"	"	261	3 200	206.4
44	25¼	1.00	0.0625	"	"	128	3 215	127.3
45	32¾	1.00	0.0625	"	"	113	3 650	111.1

is shown the strain diagram used in determining the location of the neutral axis. Fig. 17 is for illustration, and Fig. 18 is a heavy-load stress diagram for purposes of discussion.

Let E_c = the modulus of elasticity of cinder concrete;
" E_s = " " " " " mild steel;
" f_c = " extreme fiber stress on cinder concrete (working);
" f_s = " unit allowed stresses per square inch on mild steel;
" a = " area of reinforcing steel, in square inches, or in pounds per square foot;
" y_1 = " distance from top of slab to neutral axis, or
" y_2 = " " " neutral axis to center of gravity of steel;
" d = " distance from center of gravity of steel to top of slab;
" T = " total thickness of the slab; here taken as $d + 1$.

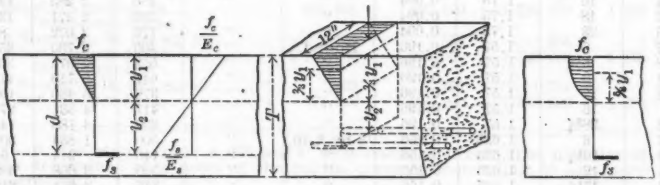


FIG. 16. FIG. 17. FIG. 18.

Referring to the diagram at the right of Fig. 16, it is evident that the maximum strain on the extreme fibers is $\frac{f_c}{E_c}$ and that the strain on the steel is $\frac{f_s}{E_s}$. Both these strains being the bases of two similar triangles, they are proportional to their altitudes, or $\frac{f_c}{E_c} : \frac{f_s}{E_s} = y_1 : y_2$; whence, $y_2 = y_1 \frac{f_s}{f_c} \times \frac{E_c}{E_s}$. If, $\frac{f_s}{f_c} = n$, and $\frac{E_c}{E_s} = m$, then $y_2 = n\,m\,y_1$; but, $d = y_2 + y_1$; hence,

$$y_1 = \frac{d}{n\,m + 1} \qquad (1)$$

Tabulation of Values for y_1.—As there may be differences of opinion as to the proper ratio of the moduli of cinder concrete and steel, the writer has prepared Table 5, giving values for y_1 (the dis-

tance from the top of the slab to the neutral axis) for different moduli, and for different ratios of unit stresses of extreme fiber on the concrete, and tension in the steel.

TABLE 5.—DISTANCE FROM EXTREME FIBER OF CONCRETE TO NEUTRAL AXIS, IN TERMS OF d.

$\frac{E_s}{E_c}$ where $E_s = 30\,000\,000$ and E_c ranges from 800 000 to 2 500 000	y_1 when $\frac{f_s}{f_c} = 80$ i. e., $\frac{16\,000}{200}$ or $\frac{20\,000}{250}$	y_1 when $\frac{f_s}{f_c} = 64$ i. e., $\frac{16\,000}{250}$ or $\frac{20\,000}{312.5}$	y_1 when $\frac{f_s}{f_c} = 53.3$ i. e., $\frac{16\,000}{300}$ or $\frac{20\,000}{375}$	y_1 when $\frac{f_s}{f_c} = 49.3$ i. e., $\frac{16\,000}{325}$ or $\frac{20\,000}{406}$	y_1 when $\frac{f_s}{f_c} = 32$ i. e., $\frac{16\,000}{500}$ or $\frac{20\,000}{625}$	y_1 when $\frac{f_s}{f_c} = 24.6$ i. e., $\frac{16\,000}{650}$ or $\frac{20\,000}{810}$
37.5	0.32d	0.37d	0.41d	0.43d	0.54d	0.60d
33.3	0.29d	0.34d	0.39d	0.40d	0.51d	0.57d
30.0	0.27d	0.32d	0.36d	0.38d	0.48d	0.55d
27.3	0.26d	0.30d	0.34d	0.36d	0.46d	0.52d
25.0	0.24d	0.28d	0.32d	0.34d	0.44d	0.50d
23.1	0.22d	0.27d	0.30d	0.32d	0.42d	0.48d
21.4	0.21d	0.25d	0.29d	0.30d	0.40d	0.47d
20.0	0.20d	0.24d	0.27d	0.29d	0.38d	0.45d
18.8	0.19d	0.23d	0.26d	0.28d	0.37d	0.43d
17.6	0.18d	0.22d	0.25d	0.26d	0.36d	0.42d
16.7	0.17d	0.21d	0.24d	0.25d	0.34d	0.40d
15.8	0.17d	0.20d	0.23d	0.24d	0.33d	0.39d
15.0	0.16d	0.19d	0.22d	0.23d	0.32d	0.38d
14.3	0.15d	0.18d	0.21d	0.23d	0.31d	0.37d
13.6	0.15d	0.18d	0.20d	0.22d	0.30d	0.36d
13.0	0.14d	0.17d	0.20d	0.21d	0.29d	0.35d
12.5	0.14d	0.16d	0.19d	0.20d	0.28d	0.34d
12.0	0.13d	0.16d	0.19d	0.20d	0.27d	0.33d

In arriving at the ratio of moduli, it was assumed that the modulus of elasticity of mild steel was about 30 000 000, and that the modulus of elasticity of cinder concrete varied from about 800 000 to 2 500 000. y_1 is given for this range of moduli and for extreme fiber stresses on the concrete of 200, 250, 300, 500, and 650 lb. per sq. in. Thus, it will be found that Table 5 will cover values of y_1 for the ordinary assumptions for stone concrete, as well as for cinder concrete.

Thickness of Concrete.—Having the values of y_1, the carrying capacity of reinforced cinder concrete slabs for varying spans and varying loads can be tabulated.

Let W = total floor load, in pounds per square foot;
" L = span of slab between supports, in feet;
" M = bending moment from loading;
" R = resisting moment of slab.

Then, the bending moment (for simply supported slabs), in inch-pounds, coming on the slab is,

$$M = \frac{12\,W\,L^2}{8},$$

the resisting moment (Figs. 16 and 17) is:

$$R = M = 12 \times \frac{y_1}{2} f_c \left(\frac{2}{3} y_1 + y_2\right),$$

$$\text{hence, } \frac{W\,L^2}{4} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

For the outside spans of continuous slabs:

$$\frac{W\,L^2}{5} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For slabs continuous throughout:

$$\frac{W\,L^2}{6} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

If $\frac{E_s}{E_c} = 30$, that is, $\frac{30\,000\,000}{1\,000\,000}$, and $\frac{f_s}{f_c} = 80$, that is, $\frac{16\,000}{200}$, then, for slabs simply supported (see values of y_1 and y_2 in Table 5):

$$\frac{W\,L^2}{8} = 24.57\,d^2, \text{ and}$$

$$d = 0.0713\,L\sqrt{W} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$T = d + 1 \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Equations 5 and 6 are for the conditions shown in Table 8.

The values for d (and for T), the thickness of concrete slabs given in other tables, were derived by substituting the proper values of y_1 and y_2, in terms of d, in Equations 2 and 4, and reducing to Equations 5 and 6.

Of course, the thicknesses of the slabs, quantities of reinforcement, and carrying capacities are based on the relative quantities of concrete and steel. An excess of concrete or an excess of steel will alter the position of the neutral axis (Equation 1) and the carrying capacity.

In these calculations the working values of f_c, etc., are about one-quarter of the ultimate; that is, at a point in the stress-strain

TABLE 6.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{300} = 53.3$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.36\,d$; $y_2 = 0.64\,d$, FROM TABLE 5.

Span, in feet.	Total Uniform Loads, in Pounds per Square Foot.															
	120		150		175		200		225		250		300		350	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.8	0.39	4.1	0.43	4.4	0.47	4.6	0.50	4.8	0.52	5.1	0.55	5.4	0.61	5.8	0.66
5.5	4.1	0.43	4.5	0.48	4.7	0.51	5.0	0.55	5.2	0.58	5.5	0.62	5.9	0.68	6.3	0.73
6.0	4.4	0.47	4.8	0.52	5.1	0.55	5.4	0.61	5.6	0.63	5.9	0.68	6.3	0.73	6.8	0.80
6.5	4.7	0.51	5.1	0.57	5.4	0.61	5.7	0.65	6.0	0.69	6.3	0.73	6.8	0.80	7.2	0.86
7.0	4.9	0.54	5.4	0.61	5.7	0.65	6.1	0.70	6.4	0.75	6.7	0.79	7.2	0.86	7.7	0.92
7.5	5.2	0.58	5.7	0.65	6.1	0.70	6.4	0.75	6.8	0.80	7.1	0.84	7.7	0.92	8.2	0.99
8.0	5.5	0.62	6.0	0.69	6.4	0.74	6.8	0.80	7.2	0.86	7.5	0.90	8.1	0.98	8.7	1.06

$d = 0.0513\,l\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.138\,d$.

TABLE 7.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{250} = 64$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.32\,d$; $y_2 = 0.68\,d$, FROM TABLE 5.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.													
	120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	4.2	0.33	4.6	0.37	4.9	0.40	5.2	0.43	5.4	0.45	5.7	0.48	6.1	0.52
5.5	4.6	0.37	5.0	0.41	5.3	0.44	5.6	0.47	5.9	0.50	6.1	0.52	6.6	0.57
6.0	4.9	0.40	5.3	0.44	5.7	0.48	6.0	0.51	6.3	0.54	6.6	0.57	7.1	0.62
6.5	5.2	0.43	5.6	0.47	6.1	0.52	6.4	0.55	6.7	0.58	7.1	0.62	7.6	0.67
7.0	5.5	0.46	6.1	0.52	6.5	0.56	6.8	0.59	7.2	0.63	7.5	0.66	8.1	0.72
7.5	5.8	0.49	6.4	0.55	6.9	0.60	7.3	0.64	7.6	0.67	8.0	0.71	8.7	0.79
8.0	6.2	0.53	6.8	0.59	7.2	0.63	7.7	0.68	8.1	0.72	8.5	0.76	9.2	0.84

$d = 0.059\,l\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.102\,d$.

TABLE 8.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{200} = 80$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.27\,d$; $y_2 = 0.73\,d$, FROM TABLE 5.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.									
	120		150		175		200		225	
	T, Total thickness of slab, in inches.	W, Quantity of steel, in pounds per square foot of area.	T.	W.	T.	W.	T.	W.	T.	W.
5.0	4.9	0.27	5.4	0.30	5.7	0.32	6.0	0.34	6.3	0.36
5.5	5.3	0.29	5.8	0.33	6.2	0.36	6.5	0.38	6.9	0.41
6.0	5.7	0.32	6.2	0.36	6.7	0.39	7.0	0.41	7.4	0.44
6.5	6.1	0.35	6.7	0.39	7.1	0.42	7.5	0.45	7.9	0.47
7.0	6.5	0.38	7.1	0.42	7.6	0.45	8.1	0.49	8.5	0.52
7.5	6.9	0.40	7.5	0.45	8.1	0.49	8.6	0.52	9.0	0.55
8.0	7.2	0.43	8.0	0.48	8.5	0.52	9.1	0.56	9.5	0.58

$d = 0.0713\,l\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.068\,d$.

TABLE 9.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{300} = 53.3$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.36\,d$; $y_2 = 0.64\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.															
	120		150		175		200		225		250		300		350	
	T, Thickness of slab, in inches.	W, Quantity of steel, in pounds per square foot of area.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.
5.0	3.3	0.32	3.6	0.36	3.8	0.39	4.0	0.41	4.2	0.44	4.3	0.46	4.6	0.50	4.9	0.54
5.5	3.5	0.35	3.8	0.39	4.0	0.41	4.3	0.46	4.5	0.48	4.6	0.50	5.0	0.55	5.3	0.59
6.0	3.8	0.39	4.1	0.43	4.3	0.45	4.6	0.50	4.8	0.52	5.0	0.55	5.4	0.61	5.7	0.65
6.5	4.0	0.41	4.3	0.46	4.6	0.50	4.9	0.54	5.1	0.57	5.3	0.59	5.7	0.65	6.1	0.70
7.0	4.2	0.44	4.6	0.50	4.9	0.54	5.2	0.58	5.4	0.61	5.6	0.64	6.1	0.70	6.5	0.76
7.5	4.4	0.47	4.9	0.54	5.2	0.58	5.5	0.62	5.7	0.65	6.0	0.69	6.5	0.76	6.9	0.81
8.0	4.7	0.51	5.1	0.57	5.4	0.61	5.8	0.66	6.0	0.69	6.3	0.73	6.8	0.80	7.3	0.87

$d = 0.042\,l\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.138\,d$.

TABLE 10.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{250} = 64$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.32\,d$; $y_2 = 0.68\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.													
	120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.6	0.28	4.0	0.31	4.2	0.33	4.4	0.35	4.6	0.37	4.8	0.39	5.2	0.43
5.5	3.9	0.30	4.3	0.34	4.5	0.36	4.8	0.39	5.0	0.41	5.2	0.43	5.6	0.47
6.0	4.2	0.33	4.5	0.36	4.8	0.39	5.1	0.42	5.3	0.44	5.6	0.47	6.0	0.51
6.5	4.4	0.35	4.8	0.39	5.2	0.43	5.4	0.45	5.7	0.48	6.0	0.51	6.4	0.55
7.0	4.7	0.38	5.1	0.42	5.5	0.46	5.8	0.49	6.1	0.52	6.3	0.54	6.9	0.60
7.5	5.0	0.41	5.4	0.45	5.8	0.49	6.1	0.52	6.4	0.55	6.7	0.58	7.3	0.64
8.0	5.2	0.43	5.7	0.48	6.1	0.52	6.5	0.56	6.8	0.59	7.1	0.62	7.7	0.68

$d = 0.0482\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.102\,d$.

TABLE 11.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{200} = 80$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.27\,d$; $y_2 = 0.73\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.													
	120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	4.2	0.22	4.6	0.25	4.8	0.26	5.1	0.28	5.4	0.30	5.6	0.32	6.0	0.36
5.5	4.5	0.24	4.9	0.27	5.2	0.29	5.5	0.31	5.8	0.33	6.0	0.34	6.5	0.38
6.0	4.8	0.26	5.3	0.30	5.6	0.32	5.9	0.34	6.2	0.36	6.5	0.38	7.0	0.41
6.5	5.1	0.28	5.6	0.32	6.0	0.34	6.3	0.36	6.7	0.39	7.0	0.41	7.5	0.45
7.0	5.4	0.30	6.0	0.34	6.4	0.37	6.7	0.39	7.1	0.42	7.4	0.44	8.0	0.48
7.5	5.8	0.33	6.3	0.36	6.8	0.40	7.2	0.43	7.5	0.45	7.9	0.47	8.5	0.52
8.0	6.1	0.35	6.7	0.39	7.1	0.42	7.6	0.45	8.0	0.48	8.3	0.50	9.0	0.55

$d = 0.058\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.068\,d$.

TABLE 12.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{500} = 32$; $\frac{E_s}{E_c} = 15$; $y_1 = 0.32\,d$; $y_2 = 0.68\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.															
	150		175		200		225		250		300		350		400	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.1	0.43	3.3	0.47	3.4	0.49	3.6	0.53	3.7	0.55	4.0	0.61	4.2	0.65	4.4	0.69
5.5	3.3	0.47	3.5	0.51	3.6	0.53	3.8	0.57	4.0	0.61	4.3	0.67	4.5	0.71	4.7	0.76
6.0	3.5	0.51	3.7	0.55	3.9	0.59	4.1	0.63	4.2	0.65	4.6	0.73	4.8	0.78	5.1	0.84
6.5	3.7	0.55	3.9	0.59	4.1	0.63	4.3	0.67	4.5	0.71	4.8	0.78	5.2	0.86	5.4	0.90
7.0	4.0	0.61	4.1	0.63	4.4	0.69	4.6	0.73	4.8	0.78	5.1	0.84	5.5	0.92	5.8	0.98
7.5	4.2	0.65	4.4	0.69	4.6	0.73	4.8	0.78	5.0	0.82	5.4	0.90	5.8	0.98	6.1	1.04
8.0	4.4	0.69	4.6	0.73	4.8	0.78	5.1	0.84	5.3	0.88	5.7	0.96	6.1	1.04	6.4	1.10
8.5	4.6	0.73	4.8	0.78	5.1	0.84	5.3	0.88	5.6	0.94	6.0	1.02	6.4	1.10	6.8	1.18
9.0	4.8	0.78	5.0	0.82	5.3	0.88	5.6	0.94	5.8	0.98	6.3	1.08	6.8	1.18	7.1	1.25
9.5	5.0	0.82	5.3	0.88	5.6	0.94	5.8	0.98	6.1	1.04	6.6	1.14	7.1	1.25	7.5	1.33
10.0	5.2	0.86	5.5	0.92	5.8	0.98	6.1	1.04	6.4	1.10	6.9	1.20	7.4	1.31	7.8	1.39

$d = 0.34\,l\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.204\,d$.

curve where the modulus of elasticity of concrete is supposed to be constant. If calculations are made for values about the elastic limit, then, according to some investigations, the moduli are variable in the concrete from the extreme fiber to the neutral axis, and the stress diagram would be somewhat like that shown in Fig. 18. With the latter assumption, the resisting moment is $12 \times f_c \times \frac{2}{3} y_1 \left(\frac{5}{8} y_1 + y_2\right)$, instead of $12 \times f_c \times \frac{y_1}{2} \left(\frac{2}{3} y_1 + y_2\right)$.

Quantity of Steel Reinforcement.—The force from the concrete (12 in. wide) resisting the bending moment due to the loading is $12 \times f_c \times \frac{y_1}{2}$; this belongs to a couple, and must equal the resistance of $f_s \times a$ in the steel. Hence, $a = 6 \frac{f_c}{f_s} y_1$, in square inches, or, in pounds per square foot,

$$a = 20.4 \frac{f_c}{f_s} y_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Substituting the values of y_1 and $\frac{f_c}{f_s}$ found in connection with each table, the quantity of reinforcement for each thickness of slab is derived.

TABLE 13.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{650} = 24.6$; $\frac{E_s}{E_c} = 15$; $y_1 = 0.38\,d$; $y_2 = 0.62\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.															
	150		175		200		225		250		300		350		400	
	T., Total thickness of slab, in inches.	*W.*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	2.7	0.54	2.9	0.60	3.0	0.63	3.1	0.66	3.2	0.69	3.4	0.76	3.6	0.82	3.8	0.88
5.5	2.9	0.60	3.0	0.63	3.2	0.69	3.3	0.72	3.4	0.76	3.7	0.85	3.9	0.91	4.1	0.98
6.0	3.1	0.66	3.2	0.69	3.4	0.76	3.5	0.79	3.7	0.85	3.9	0.91	4.1	0.98	4.4	1.07
6.5	3.2	0.69	3.4	0.76	3.6	0.82	3.7	0.85	3.9	0.91	4.2	1.01	4.4	1.07	4.6	1.13
7.0	3.4	0.76	3.6	0.82	3.8	0.88	3.9	0.91	4.1	0.98	4.4	1.07	4.7	1.17	4.9	1.23
7.5	3.6	0.82	3.8	0.88	4.0	0.94	4.1	0.98	4.3	1.04	4.6	1.13	4.9	1.23	5.2	1.32
8.0	3.7	0.85	4.0	0.95	4.2	1.01	4.3	1.04	4.5	1.10	4.9	1.23	5.2	1.32	5.5	1.42
8.5	3.9	0.91	4.1	1.00	4.4	1.07	4.6	1.13	4.8	1.20	5.1	1.29	5.4	1.39	5.8	1.51
9.0	4.1	0.98	4.3	1.04	4.6	1.13	4.8	1.20	5.0	1.26	5.4	1.39	5.7	1.48	6.0	1.57
9.5	4.3	1.04	4.5	1.10	4.8	1.20	5.0	1.26	5.2	1.32	5.6	1.45	6.0	1.57	6.3	1.67
10.0	4.4	1.07	4.7	1.17	4.9	1.23	5.2	1.32	5.4	1.39	5.8	1.51	6.2	1.64	6.6	1.76

$d = 0.028\,l\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.315\,d$.

Excessive Concrete or Steel.—To note the effect of excessive concrete or of excessive reinforcement on the carrying capacity, observe y_1, Equation 1. Assuming the working stress in the steel to be constant, y_1, in Table 5, increases with an increase in extreme fiber stress and decreases with an increase in the modulus of the concrete. Some designers use cinder concrete with an excess of steel in order to keep the slab as thin as possible.

Example 1.—Suppose we are limited to a working extreme fiber stress of 250 lb. per sq. in., are designing for a total floor load of 150 lb. per sq. ft. on a 6 ft. 0-in. span (simply supported slabs), but wish to limit the thickness of the slab to 4 in. (3 in. effective depth).

In this case, the extreme fiber stress being fixed at 250 lb. per sq. in., and the ratio of moduli being assumed as constant at 30, the variable to be sought in determining y_1 is the stress in the steel. Suppose we use sufficient reinforcement to bring y_1 midway between the center of the steel and the top of the slab.

Referring to Equation 1, and substituting values for d (3 in.), m (30), and y_1 ($1\frac{1}{2}$), we have $\frac{3}{2} = \frac{3}{\frac{n}{30} + 1}$; but $n = \frac{f_s}{f_c} = \frac{f_s}{250}$; hence, $f_s = 7\,500$.

Therefore, in order to bring the neutral axis midway in the effective depth of the slab and not exceed 250 lb. per sq. in. in extreme fiber stress, the reinforcement must not be strained beyond an amount given by a stress of 7 500 lb. per sq. in. The weight of reinforcement in this case will be (Equation 7): $a = 1.02$ lb. per sq. ft. Now, with steel stressed to 16 000 lb. per sq. in., as assumed in Table 7, the total thickness of the slab would have been 5.3 in., but the quantity of steel would then have been only 0.44 lb. per sq. ft. Hence, 0.58 lb. per sq. ft. is required to save 1.3 in. in thickness of cinder concrete. With reinforcing steel at $2\frac{1}{2}$ cents per lb., the 0.58 lb. of excess steel will cost 1.45 cents per sq. ft.; and the saving effected will be 1.3 in. of concrete.

Example 2.—Suppose we have a 6 ft. 0-in., simply supported slab, and are using a cinder concrete having a working capacity of 250 lb. per sq. in. of extreme fiber stress (Table 7); and, for some reason, we wish to use a certain reinforcement, a little light in area, to take the tension at 16 000 lb. per sq. in. We are not allowed to exceed the 16 000 lb., and, therefore, must thicken the slab, to more than the normal thickness given in Table 7; that is, we must use an excess of concrete.

If the total floor load is 150 lb. per sq. ft., we find in Table 7 that the slab will be 5.3 in. thick, and the reinforcement will weigh 0.44 lb. per sq. ft. of floor.

Assume that the reinforcement we wish to use weighs only 0.36 lb. per sq. ft. We see by Equation 7 that a decrease in the section of the reinforcement may be effected by a decrease in the stress on the extreme fiber of the concrete. Therefore, looking in Table 8, which has a less extreme fiber stress, we find that, with a slab 6.2 in. thick, the reinforcement is 0.36 lb. per sq. ft. of floor.

If a less quantity of reinforcement had been desired, then calculations using lower extreme fiber stresses, similar to the calculations used for making these tables, would have been necessary.

Stresses in Approved Floor Slabs.—The location of the neutral axis for any particular thickness of slab is a function of the relative stresses in the materials, and of the moduli of elasticity. Referring to Fig. 7, assume that the ratio of moduli remains constant at 30, and that, $\frac{f_s}{f_c} = 64$, that is $\frac{16\,000}{250}$ or $\frac{20\,000}{312\frac{1}{2}}$, and that the slab tested is continuous; we find in Table 10 (the table using these assumptions) that the neutral axis is $0.32d$ from the top of the slab when the concrete and steel are proportioned for these stresses. For a 4-in. slab, therefore, the neutral axis would be 0.96 in. from the top of the slab. In Table 10, a normal 4-in. slab is good for a total floor load of 150 lb. per sq. ft. on a 5 ft. 0-in. span, with a reinforcement of 0.31 lb. per sq. ft.; but in this 4-in. slab, of 8 ft. 0-in. span (Fig. 7), as approved, No. 5 wires at 3-in. centers are used. This makes the actual reinforcement about 50% greater than the normal reinforcement for Table 10.

This relative excess of steel (reducing the comparative strain coming on the reinforcement) will lower the neutral axis below that given in Table 10. The strain on the steel, $\frac{f_s}{E_s}$, therefore, is in this case, $\frac{2}{3} \times \frac{16\,000}{30\,000\,000}$ (Fig. 16). Then, the neutral axis, on account of this excess of steel, will be located $0.41d$ or 1.23 in. from the top of the slab—instead of 0.96 in. where there is no excess of steel.

According to the ruling of the New York City Building Department, the 4 in. of cinder concrete weighs 32 lb. per sq. ft. of area. Therefore, the approval of this floor construction is for a total floor load of $250 + 32 = 282$ lb. per sq. ft.

Therefore, the bending moment, from this total load (if the slab is called continuous) is $\frac{W L^2}{1}$, or, 18 048 in-lb.

This is equal to the resisting moment (one force of the couple being $12\ f_c \frac{1.23}{2}$) having a lever arm, $\frac{2}{3} \times 1.23 + 1.77$ in., or $19.11 \times f_c$, in inch-pounds. Therefore, $f_c = 944$ lb. per sq. in., which

is the extreme fiber stress on the concrete. The area of the reinforcement being 0.134 sq. in., $f_s = 52\,000$ lb. per sq. in. in the steel.

If, instead of considering the slab continuous, we take an end slab,* as is done with stone concrete, we have a moment of $\frac{12\ W\ L^2}{10}$, or 21 658 in-lb. In this case the extreme fiber stress will be 1 133 lb. per sq. in., and the stress on the steel, 62 000 lb. per sq. in.

If we take the condition of reinforcement simply laid over the supports, as shown in Figs. 14 and 15, then the slab is simply supported at the ends, and the load moment becomes $\frac{12\ W\ L^2}{8}$, or 27 072 in-lb. This latter condition gives an extreme fiber stress of 1 417 lb. per sq. in., and a steel stress of 78 000 lb. per sq. in.

Of course, under such abnormal stresses the modulus of elasticity for cinder concrete, probably, does not remain constant, and the resistance diagram may assume a shape similar to Fig. 18. This would slightly modify the foregoing deductions. However, even if an indefinite number of assumptions were made regarding the ratio of moduli of elasticities of wire and of cinder concrete, ratios of stresses in concrete and steel, and the variable support of slabs, etc., the foregoing deduced stresses would still be found to represent approximately correct conditions.

Without further attempt to lengthen this inquiry (by considering an indefinite number of cases and conditions), the writer would call attention to the comparison of the thickness of stone concrete slabs with approved cinder concrete slabs under similar conditions. It is found that, for the same conditions as the approved cinder concrete slab (Fig. 7) discussed herein, the stone concrete would be required to be about 1 in. thicker than the cinder concrete.

Recommendations.—

1.—The practice of testing cinder concrete, arbitrarily, under the most favorable conditions, and then recommending the results of these tests to be used under the most unfavorable conditions which may exist in a building, is dangerous and entirely unnecessary.

2.—Unit stresses for the design of reinforced cinder concrete are of more importance for its proper use than the unit stresses

* Progress Report, Special Committee on Concrete and Reinforced Concrete, *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 385.

used for reinforced stone concrete, because of the greater variableness of the former.

3.—There are, at present, sufficient precedents for choosing some unit stresses for the use of reinforced cinder concrete slabs in each locality where this material is being used.

4.—It is practical, and of little cost for each city, to establish ultimately proper working stresses for reinforced cinder concrete, based on tests with the materials used in their locality.

5.—With cinder concrete used according to proper working stresses, the construction will be removed from the realm of mystification and of misrepresentation to the field of Civil Engineering.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION.

By W. C. Hammatt, M. Am. Soc. C. E.

To be Presented May 20th, 1914.

Summary.

This paper describes suburban and country road conditions in California and the progress in road building in that State during the past few years. The State contains plentiful supplies of good road materials, rock, gravel, asphaltic oils, etc., and there are several cement factories.

The water-bound and oiled macadam roads are described and also concrete and asphaltic pavements; and details are given as to the good and bad features of the bases and the wearing surfaces. Some costs of asphaltic pavements are included.

With exceptional advantages in regard to road materials, California is, nevertheless, in its infancy in the study of road construction. Until about six years ago, practically no permanent road surfacing had been done outside of the municipalities. In the different counties some of the more important roads had been graveled, and a great many of the county roads had been oiled, but these measures

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were taken more for the purpose of maintenance than as permanent construction work. The graveling was generally done during the rainy season in order to make the roads passable, and as the gravel was gradually worked down into the mud it was replaced by more until the depth of penetration of the surface water was reached, when the road became a good gravel highway. The oiling was generally done in dry weather, and the oil and dust formed a thin crust on the surface which was broken through by heavy traffic, and if not promptly repaired the road was soon impassable.

The southern counties of the State, which have ever been the leaders in public improvements, had expended some funds prior to the general awakening in 1908. These counties developed certain classes of cheap construction (to be described later), which, for a time, gave them an enviable reputation for roads. In 1908, however, the northern counties awakened to the need of better roads, and bond issues were passed in several of these counties for the purpose of putting permanent surfaces on their principal roads. The idea of a State highway system was developed in 1911, and, in 1912, a State Highway Commission was created and a bond issue of $18 000 000 was voted for the construction of a system of State highways.

Materials.

California is rich in many of the materials which go to make up the modern road. The Coast Range is prolific in trap rock. The Sierras are composed mainly of granite, though the foot-hills are of sandstone and shale; there are, however, in the foot-hills, a few quarries of good road rock. Gravel of a good quality for either road material or concrete, is found within easy distance of nearly every locality. There are four large companies manufacturing cement in the region of San Francisco Bay; mineral oil, with an asphaltum base, is obtained in eight different fields, distributed over about one-third of the State; and the largest refinery in the United States, producing asphaltum and asphaltic road oils, is located on San Francisco Bay.

Types of Road Construction.

Only the types of road surface used on country and suburban roads will be discussed in this paper, and each type will be taken up and described separately.

Water-Bound Macadam.—This type of pavement needs no description; it has been laid extensively in California, particularly about the centers of population. In many cases it has been laid to a finished thickness of as little as 4 in., in which case the coarsest size of rock is omitted, and the macadam is laid in two sizes only. It has been found that trap rock makes the best pavement, and that some of the softer varieties have the best cementing qualities. In fact, some of the best macadam pavements which the writer has ever seen have been laid with rock which would not pass the rattler test.

Where the sub-base has been properly prepared and drained, and the rolling has been thoroughly done, and where they have had the necessary sprinkling, macadam roads have stood up remarkably well even under automobile traffic. It is a deplorable fact, however, that the average road-builder underrates the necessity for care in the preparation of the sub-base.

Two facts are worthy of special note in the building of a macadam road: First, that a good macadam road cannot be built in rainy weather or when the sub-base is soft or wet from any other cause. To obtain proper cementation of the rock, there must be considerable resistance to the rolling. Second, the duration of the rolling cannot be specified in units of time per unit of area, as it depends on the nature of the rock and on various other conditions which make the judgment of the engineer in charge necessary.

Oiled Macadam.—This type of paving is peculiar to the localities producing an asphaltic oil. There is no fixed specification for its construction, but it is laid under many different ones, according to the whims of the engineer in charge. The name covers all classes of road surface in which oil is combined with road metal as a binder. The usual method of construction is to lay the road metal and roll the various layers in the same manner as for water-bound macadam, applying to each layer by a sprinkler a certain specified quantity of asphaltic road oil. In constructing this pavement, the main difference of opinion among road builders is in regard to the quantity of oil to be applied.

As a matter of fact, a very small proportion of the total area of oiled macadam pavement constructed has been successful. Under heavy traffic it ruts and waves very badly. It is the writer's opinion that, even under the best conditions, this pavement has little justifica-

tion. Oil has more lubricating than binding properties, and its introduction into a macadam pavement has a tendency to destroy the stability of the metal. The successful examples of pavements of this class have been either in localities where climatic conditions are such as to evaporate the more volatile parts of the oil, or where the application has been made in such a manner as to achieve the same result. Oiled macadam roads which have become so rutted as to be almost impassable, have been converted into fairly good pavements by scarifying and re-rolling, thereby aerating the oil and causing the evaporation of the lighter constituents.

Under this heading should properly come a patented pavement known as Petrolithic paving, which has been developed in the Southwest, and tried somewhat in other localities. It consists essentially of the incorporation with the natural soil of a certain definite proportion of rock and a certain quantity of oil, and the tamping of the mass into a crust, or pavement, by a roller containing spike-like projections which tamp the mixture from the bottom up. This pavement has been used with some success in localities where the soil is of a sandy rather than a clayey character, and where the summer temperature is high.

The cost of oiled macadam pavements, 6 in. in finished thickness, is about 75 cents per sq. yd., and that of Petrolithic pavements of the same depth about 60 cents.

Concrete Pavements.—Very little straight concrete pavement has been laid in California without a protective wearing surface. The writer knows of one experimental piece which has been laid with expansion joints, but it has been in use too short a time to be the source of any information at present. It was put down in two courses, the finish course being laid before the base course had set. The pavement is 6 in. thick, and cost about 90 cents per sq. yd.

In order to eke out an insufficient bond issue as far as possible, the State Highway Commission specified as the standard pavement, a 4-in. concrete base with a 1½-in. asphaltic wearing surface, and ordered that the base alone should be constructed at first, and either covered with a temporary coating of oil and screenings, or left unprotected until additional funds were available. Several hundred miles of this base have been laid, a large part of which has broken up into a series of slabs by shrinkage cracks, some of them being more

than 2 in. wide. One stretch, to the writer's knowledge, has gone to pieces by surface wear, but possibly this is due to poor concrete work which might have been prevented by better inspection. The cost of this base, inclusive of grading and what little protective coating has been used, has averaged 90 cents per sq. yd.

Asphaltic Pavements.—As might be expected from the fact that asphalt of a high grade is manufactured in California, the cities and suburban districts contain a high proportion of asphaltic pavements. The standard San Francisco pavement is 2 in. of sheet-asphalt on 6 in. of concrete. In the smaller suburban cities, this is modified to 1½ in. of sheet-asphalt on 4 in. of concrete. In late years sheet-asphalt is being replaced by so-called asphalt macadam, under the various specifications of Warrenite, Topeka specifications, etc. The concrete base often has asphalt macadam substituted therefor, and there is now a tendency toward the macadam base for asphaltic pavements.

As opinions differ largely as to the respective merits of the various classes of bases and wearing surfaces, the writer will merely give the results of his experience and his opinions derived therefrom.

Bases.

(*a*).—The concrete base has the disadvantage of a high shrinkage in setting and a large coefficient of expansion. The former causes the base to crack into slabs in setting and the consequent destruction of its monolithic quality. The latter causes a movement after the wearing surface, and, consequently, cracks in the latter. Another fault, which is one of general practice in construction rather than in the materials of construction, is that the concrete is generally laid in such a manner that the surface is gently undulating rather than absolutely uniform. This causes the wearing surface to be of varying thickness and subject to unequal movement, as will be described later under that heading. This can be obviated, however, by trimming the concrete base with a template while laying.

This base is the one most commonly used, due to a popular prejudice in its favor. Most laymen and many engineers cannot get away from the idea that a rapid chemical reaction will produce a substance more durable and serviceable than rock combined according to the laws of gravity and stability.

(*b*).—The asphaltic macadam base has not given good results, for the following reasons: It has no stability, but is subject to the same faults as the too-thick asphaltic wearing surface which will be described later.

In addition, asphalt in contact with soil has a tendency to promote vegetable growth, and many pavements of this class have been heaved up and broken from root growth, where, prior to the placing of the pavement, such growth did not exist.

(*c*).—A water-bound macadam base is the best for supporting an asphaltic wearing surface, for various reasons. It has a stability due to the seating of its component parts, which is not dependent on any chemical action. It has no movement under temperature changes. It is not rigid, and, therefore, does not give the destructive reaction to impact on the surface of the asphaltic pavement, which is a feature of the concrete base. The surface is free from depressions which vary the thickness of the asphaltic wearing surface.

Wearing Surface.

(*a*).—The limits of efficacious thickness of the wearing surface are quite small. The wearing surface must be thick enough to be held in position by its own inertia, as it has but small cohesion to the base; and it must be so thin that the compression of rolling shall be communicated throughout its entire thickness. It is inadvisable to make the wearing surface less than 1½ in., or more than 2 in., in thickness. Where the asphaltic wearing surface is laid to such a thickness that the compression is not communicated throughout the entire depth, the compression is not uniform, and the subsequent adjustment, which always takes place in an asphaltic mixture of insufficient density, causes inequalities in the surface which are starting points for failure by waving.

(*b*).—The quality of the pavement and the roughness of the surface are not dependent to any great extent on the size of the coarse aggregate. The best and most durable mixtures are those containing the highest proportion of fine aggregate of 80 to 200 mesh. If the mixture is laid hot enough, and the compression is properly done, the fine material, or "mortar", will come to the surface and thoroughly fill all the voids in the coarse aggregate, which then merely serves to decrease the quantity of asphalt necessary. The only way of obtaining

the rough surface so much desired by some, is either to have insufficient fine material to fill the voids, or to lay the mixture too cold or with insufficient compression. In each case, the wearing surface will be open and spongy. Also, if the coarse aggregate projects above the general surface of the pavement, the stones are crushed or rolled by steel tires passing over them, leaving voids where disintegration may start.

The costs of the various types of asphaltic pavement are as follows:

2-in sheet-asphalt, on a 4-in. concrete base.....$1.50 per sq. yd.
1½-in. Topeka specification, asphaltic wearing surface on a 4-in. concrete base.....$1.15 per sq. yd.
2-in. Topeka specification, asphaltic wearing surface on a 4-in. macadam base.....$1.10 per sq. yd.

The laying of an asphaltic wearing surface on an old macadam pavement, with no other preparation than the cleaning off of the fine material, has been done with great success. The cost of a 1½-in. Topeka specification wearing surface laid on old macadam is about 50 cents per sq. yd. All the costs given are based on labor and material costs prevailing in the bay counties about San Francisco.

Another matter worthy of note is the tendency of municipal governments in California to require street improvement work in the outlying districts to follow the same lines as in the thickly settled parts. Where the property is in large tracts, the villas have their own driveways, so that there is no occasion for teams or vehicles to stop in front of the property, nevertheless most of the municipalities require the placing of cement curbs and sidewalks, and the paving of the roadway from curb to curb, as in business blocks, thus spoiling the possibilities of architectural landscape work.

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HUACAL DAM, SONORA, MEXICO.

By H. Hawgood, M. Am. Soc. C. E.

To be Presented May 20th, 1914.

SUMMARY.

This paper describes the methods used in the design and construction of a concrete arch dam dependent on arch action for its stability. The dimensions of the dam are: crest length of curved structure, 140 ft.; radius, 76 ft.; maximum height, 100 ft.

The futility of attempting a determination of stresses by pure mathematical processes is discussed. It is pointed out that the lack of definite knowledge as to the behavior, under stress, of the rock in which the ends and base of the dam are intrenched, and to which it is joined, inhibits the application of pure mathematics, and the use of the simple formula for a thin cylinder is sustained.

For general information, and for comparative purposes, Table 1 gives the characteristic dimensions and stresses of numerous pure arch dams in various parts of the world.

The cost of the work is given, and there is also a description of the pipe line, conveying the impounded water from the dam to the place of use, together with its cost.

The works were executed in a difficult country, with impossible wagon roads, where the cement, sand, and some other materials had to be packed on animals, and where labor was disturbed by a state of war.

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The concrete dam described in this paper is an arched structure dependent on arch action for its stability. The use of the arch for dams and dependence on it for stability, is old; how old is not susceptible of proof, but certainly more than a century. Some time prior to 1800 was built the Meer Allum Dam, at Hyderabad, India, a multiple-arch structure composed of a succession of brick arches, varying from 40 to 82 ft. in radius, with an aggregate length of about ½ mile and a water depth of about 40 ft. A more recent Indian example is a dam 60 ft. high, also of the multiple-arch type, at Alwar, Rajputana. In Europe the Zola Dam, near Aix, France, of a radius of 158 ft. and a height of about 120 ft., dates from 1843.

In more recent years many arch dams have been built, notably in New South Wales and South Australia. Their characteristic dimensions are given on Table 1.

In the United States, the Bear Valley Dam, Southern California, built in 1886, has in the past attracted much attention by the boldness of its design, and yet its audacity is equalled by the less known Upper Otay Dam of the San Diego Water Supply, built in 1900. The granite masonry of the Bear Valley Dam was subjected to compressive stresses of about 61 tons per sq. ft., and the concrete of the Upper Otay Dam is subjected to stresses of similar intensity. The Otay Dam is an instructive illustration of the stresses which can be sustained successfully by a horizontal arch of Portland cement concrete, mixed 1:2.1:3.4, when founded and abutting on strong, hard rock. The advisability, however, of imposing such high stresses may well be questioned.

It is of interest to note that the old Bear Valley Dam, after doing duty for 27 years, has at last survived its usefulness. It has been superseded, and at high-water stages is submerged by the water impounded by a higher dam of the inclined multiple-arch type, designed and built by J. S. Eastwood,* M. Am. Soc. C. E.

The writer has not found any record of the failure of a true arch dam; if any have failed, it would be well to have all the facts brought to light.

It will be noted that the stresses sustained by the various structures have an extremely wide range, varying from a minimum of 10.6 tons to a maximum of 61 tons per sq. ft., or from 147 lb. to 870 lb. per sq. in.

* *Engineering News*, December 25th, 1913, Vol. 70, p. 1284.

After study of the available local materials and labor conditions, a stress of 16.3 tons per sq. ft., 226 lb. per sq. in., was adopted for the Huacal Dam. If labor experienced in concrete work had been available at Nacozari, a higher stress might have been considered. The most vigorous supervision cannot entirely eliminate the errors of uninformed labor; foremen, however competent, cannot be everywhere at once, and it is prudent to make allowance for probable inferior quality of the labor, an important factor in the proper placing of concrete.

The physical characteristics of the coarse aggregate enter largely into the question of allowable compressive stresses.

General.

The Huacal Dam was constructed during 1911-12 for the Moctezuma Copper Company, Nacozari, Sonora, Mexico (Phelps, Dodge and Company, New York), from the designs and under the general direction of the writer.

The purpose of the dam is to impound storm waters and create a gravity supply for the milling operations of the Copper Company. The water supply prior to the construction of the dam was obtained by pumping from the gravel beds of the Rio Nacozari. For a period of from 2 to 4 months of each year the waters of these gravels became so far depleted as to impose limitations on the output of the mill. The normal capacity of the mill is 400 tons of concentrates per day, and the required daily supply of water is about 1 250 000 gal. The pumping plant, now out of service, but held as an emergency reserve, consists of two 10 by 12-in. Aldrich triplex plunger pumps, electrically driven through gearing, discharging against a head of 225 ft. In round figures, the yearly cost of operating these pumps was $12 500. Decrease in cost and increase in water supply constituted the economic incentive for the dam.

Nacozari, the present southern terminus of the Ferro Carril de Nacozari, is 76.6 miles south of the International Boundary. The northern termini of the railroad are Douglas, Ariz., on the American side of the line, and Agua Prieta, on the Mexican side.

The dam is built across the canyon of Huacal Creek, about 3 miles northeast from Nacozari, at an elevation of 550 ft. above the town and 4 100 ft. above sea level. Nacozari formed the base of construction

TABLE 1.—CHARACTERISTICS

	Date of construction.	Location or name.	*R*, Radius, in feet.	*L*, Top length of segment, in feet.	Arc, in degrees.
		UNITED STATES OF AMERICA.			
a	1886.....	Bear Valley, Cal.....	335	300	51° 20′
b	1900.....	Upper Otay, Cal.....	359	350	56° 00′
c					
d	1907.....	Crowley Creek, Ore.....	70	150	55° 50′
e	1913.....	"Goodwin Dam," Stanislaus River, Cal. Twin arches, each.....	135	233	100° 00′
		MEXICO.			
	1912.....	Huacal, Sonora.....	76	140	105° 40′
		SOUTH AUSTRALIA.			
	1904.....	Barrosa.....	200	472.5	135° 20′
		NEW SOUTH WALES.			
	1897.....	Parkes.....	300	540	100° 00′
	1898.....	Costamundra.....	250	475	109° 30′
	1898.....	Tamworth.....	250	440	100° 00′
	1899.....	Wellington.....	150	350	133° 00′
	1899.....	Mudgee.....	253	498	112° 00′
	1899.....	Wollongong.....	200	344	98° 30′
	1896.....	Lithgow No. 1.....	100	162	93° 00′
	1897.....	Picton.....	120	112	53° 00′
	1898.....	Queen Charlotte Vale.....	90	113	72° 00′
	1858				
	1898	Paramatta.....	160	225	80° 00′
	1906.....	Lithgow No. 2.....	100	221	127° 00′
	1906.....	Medlow.....	60	81	78° 00′
	1907.....	Barren Jack.....	80	.....	.....
		INDIA.			
g	Prior to 1800.....	Meer Allum.....	82	178	175° 00′
		FRANCE.			
h	1843.....	Zola, Aix.....	158	205	74° 20′

AUTHORITY.

a. Bear Valley Company's Original Records, also Schuyler's "Reservoirs for Irrigation, Waterpower and Domestic Supply," pp. 246-256.
b. *Engineering News*, Vol. 46, p. 125.
c. *Engineering News*, Vol. 51, p. 337.
d. *Engineering News*, Vol. 66, p. 220.
e. *Engineering News*, Vol. 70, p. 748.

supplies. Bulky pieces were transported by wagon, over a rough mountain road some 6 miles in length, with steep gradients, the maximum load being 1 ton to four good animals. Everything else, including all cement and sand, was packed on mules and burros, with loads of 300 lb. or more to the mule, and 175 lb. to the burro. The

OF CONSTRUCTED "ARCH-DAMS".

D, Depth of water, in feet.	*T*, Thickness at *D*, in feet.	*S*, Stress, in tons per square foot.	Nature of aggregate.	Class of masonry.
48.0	8.42	61.0	Granitic.	Granite masonry.
75.0	14.00	61.0	Granitic.	Concrete.
50.0	12.00	46.7		
63.0	5.16	26.6	Lava.	Concrete.
70.0	at *D*= 61.0 ft. 16.0 "	21.4	Gravel.	Concrete.
88.5	12.83	16.3	Andesitic.	Concrete.
94.0	34.00	17.3	Gneiss.	Concrete.
37.0	13.50	29.1	Granitic.	Concrete.
45.5	12.88	28.2	Granitic.	Concrete.
62.0	21.50	23.3	Granitic.	Concrete.
48.5	10.00	23.7	Conglomerate.	Concrete.
50.0	18.00	22.4	Slate.	Concrete.
41.5	11.62	22.9	Basalt.	Concrete.
38.0	11.38	11.4	Sandstone.	Concrete.
40.0	13.62	13.8	Sandstone.	Concrete.
34.0	8.65	11.7	Quartzite.	Concrete.
46.0	15.00	15.3	Sandstone.	Concrete.
78.5	24.00	10.6	Sandstone.	Concrete.
66.0	8.96	14.4	Sandstone.	Concrete.
38.0	5.00	19.0	.....	Concrete.
39.0	8.50	11.8	.....	Brick masonry.
119.7	41.80	14.2	.....	Rubble masonry.

f. *Minutes of Proceedings*, Inst. C. E., Vol. 178.
g. *Minutes of Proceedings*, Inst. C. E., Vol. 172, p. 214.
h. Schuyler's "Reservoirs for Irrigation, Waterpower and Domestic Supply," p. 362.
The stresses, *S*, in Table 1, are computed by the cylinder formula:

$$\text{Stress, in tons (2 000 lb.) per sq. ft.} = \frac{\text{Radius (feet)} \times \text{Depth (feet)}}{32 \times \text{Thickness (feet)}}.$$

average cost of packing was about $1.02 per 2 000 lb. The cement was packed from the railroad depot, a distance of about 6 miles, and the sand from the Nacozari River, about 4 miles. The cost per ton-mile, one way, averaged about 21¾ cents.

All prices and costs are given in United States gold values.

Catchment Area.

The catchment tributary to the dam (Fig. 3) has an area of 13 sq. miles, is roughly rectangular in form, with an extreme length of 5.7 miles and a width of 3.4 miles. It is a mountainous area, broken by numerous watercourses with deeply incised channels. The slopes leading down to the channels are steep, vegetation is sparse, the soil shallow, with much bare rock, and the storm discharges are of a torrential character.

Rainfall.

The rains commence in June. May is a dry month, and the rain records are in the best form for comparative purposes when arranged by seasons running from June 1st to May 31st of the succeeding year. A rain gauge has been kept at Nacozari since May, 1910, but no definite information as to previous rainfall can be obtained.

TABLE 2.—Rainfall at Nacozari by Seasons.

Month.	1910-11. Inches.	1911-12. Inches.	1912-13. Inches.	1913-14. Inches.
June	0.51	2.66	1.17	0.08
July	4.39	3.73	4.66	4.13
August	2.32	2.25	3.59	2.77
September	0.89	4.79	0.00	
October	0.14	3.36	0.54	
November	0.33	1.38	0.83	
December	0.00	1.13	0.77	Highest rate of precipitation 3.05 in. in 3 hours, October 4th, 1911.
January	1.73	0.00	0.84	
February	1.43	0.31	4.52	
March	0.01	1.96	0.57	
April	0.60	0.20	0.53	
May	0.00	0.00	0.02	
Totals	12.35	21.77	18.04	

No gauges are maintained other than the one at Nacozari. The rainfall over the catchment area above the dam is unquestionably greater than at Nacozari, but to what extent is as yet purely a matter of estimate and conjecture.

Run-Off.

On the basis of the probable rainfall and the topographical features of the catchment area, it was estimated that the yield of a rain season such as 1910-11 would be approximately 3 000 acre-ft. (2 700 000 gal. per day). The season of 1910-11 was classed as "dry" by those familiar with the country. This opinion is supported by the rain records of

FIG. 1.—DAM SITE, LOOKING DOWN STREAM.

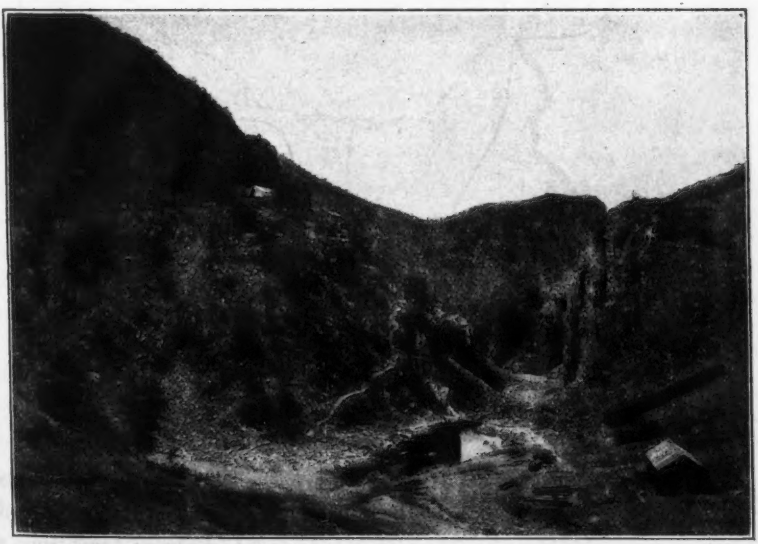
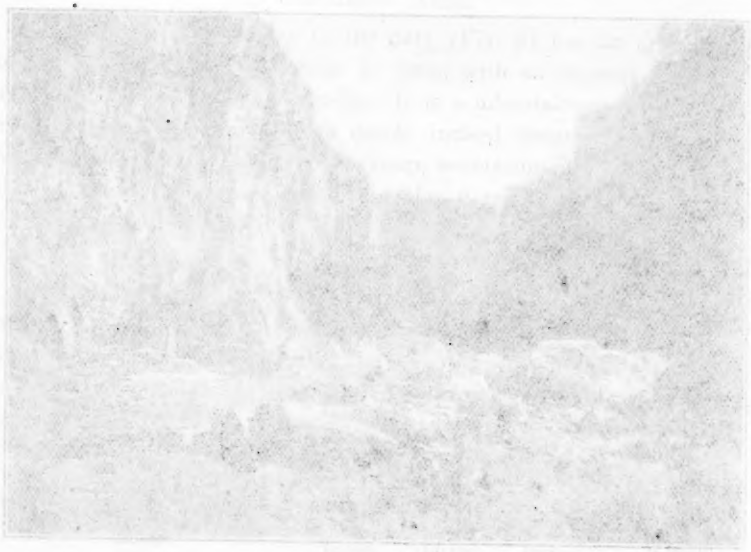


FIG. 2.—COMMENCEMENT OF WORK. LOOKING DOWN STREAM.



1911-12, 1912-13, as far as they go, but they are of too short duration to form any conclusions, and it cannot be assumed that there will not be drier seasons than 1910-11. The margin between the con-

templated use of 1 400 acre-ft. annually, or 1 250 000 gal. daily, and the 3 000 acre-ft., or 2 700 000 gal. daily run-off estimated for 1910-11 is considered ample to take care of drier seasons.

On August 14th, 1910, a storm run-off was observed lasting about 1 hour, the total discharge from which was estimated at 300 acre-ft., with an average run-off rate of 255 sec-ft. per sq. mile.

On October 5th, 1911, a flood, having a maximum rate of 2 000 cu. ft. per sec., passed, without damage, over the partly finished dam, then 40 ft. high. The total discharge during this storm was estimated to be 1 240 acre-ft., with a maximum run-off rate of 154 sec-ft. per sq. mile.

Reservoir and Dam Site.

In outline, the reservoir is T-shaped, as shown by Fig. 4. The dam and outlet of the reservoir are on the stem of the T, to each side of which the water surface at high water extends for about 1 mile.

The capacity of the reservoir at the crest level of the dam is 2 700 acre-ft., with a surface area of 93 acres. It is designed, should it be found desirable to impound more water, to install tumbling flash-boards along the crest of the dam.

At the point selected for the dam the canyon had in its natural state a bottom width of from 20 to 30 ft., and, at 90 ft. above the bottom, a width of 150 ft.

The geological formation is Andesitic. In the main, the rock is hard and close-grained. The dip of the formation approaches the vertical, and the strike is across the canyon approximately at right angles to its axis, both of which are conditions favorable to minimum leakage and stability of foundation. A seam discharging a little water was disclosed in driving the outlet tunnel and in the shaft beneath the outlet tower. During the active progress of the work it was impracticable to isolate and measure this water. In April, 1912, after the water in the reservoir had arisen against the dam to a height of about 40 ft., this spring was measured and found to yield 1.5 gal. per min. The entire flow over the rock bed of the stream at a point about 100 ft. below the dam was, on the same date, 3.58 gal. per min. This would ascribe a seepage through the dam and its foundation contact of 2.1 gal. per min. In February, 1913, 10 months later, the entire flow over the stream bed was again measured and found to have decreased to 2.83 gal. per min., the depth of water against the dam having in the interim increased from 40 to 60 ft.

The seepage through the dam and its foundation contact was not precisely segregated, but was judged not to exceed 1.0 or 1.5 gal. per

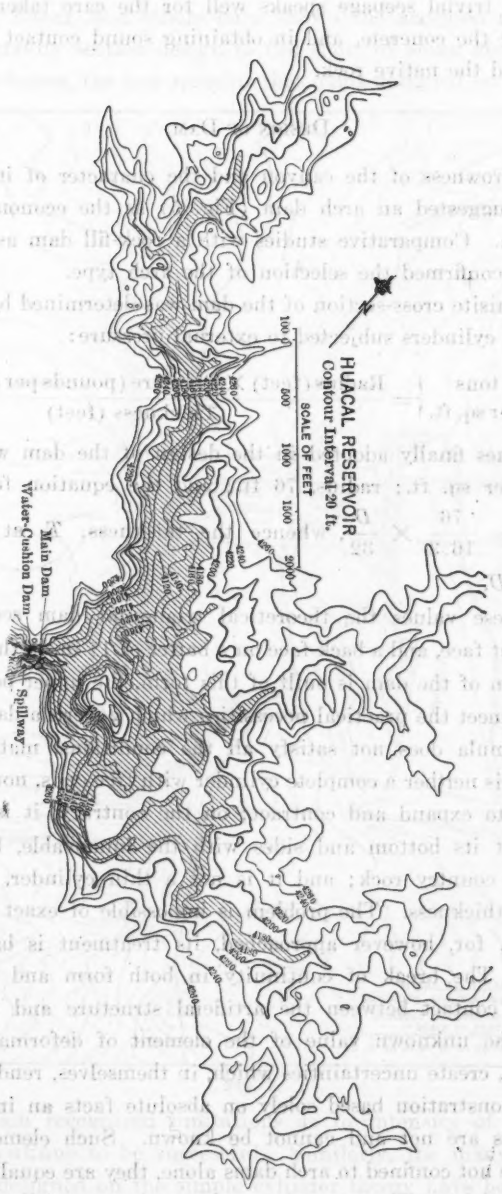

HUACAL RESERVOIR
Contour Interval 20 ft.
SCALE OF FEET
100 0 500 1000 1500 2000
Main Dam
Water-Cushion Dam
Spillway

FIG. 4.

min. This trivial seepage speaks well for the care taken in making and placing the concrete, and in obtaining sound contact between the concrete and the native rock.

Design of Dam.

The narrowness of the canyon and the character of its rock sides and floor suggested an arch dam (Fig. 5) as the economic type for the location. Comparative studies with a rock-fill dam as a practical alternative confirmed the selection of the arch type.

The requisite cross-section of the dam was determined by the simple formula for cylinders subjected to external pressure:

$$\left.\begin{matrix}\text{Stress, in tons} \\ \text{(2 000 lb.) per sq. ft.}\end{matrix}\right\} = \frac{\text{Radius (feet)} \times \text{Pressure (pounds per square foot)}}{\text{Thickness (feet)}}.$$

The values finally adopted in the design of the dam were: Stress, 16.3 tons per sq. ft.; radius, 76 ft.; and the equation for thickness became $T = \frac{76}{16.3} \times \frac{D}{32}$, whence the thickness, T, at the depth, $D = 0.146\ D$.

With these values the theoretical triangular dam section has a vertical front face, and a back face on a batter of $1\frac{3}{4}$ in. to the foot. The lower portion of the dam is built of this form, the upper portion being widened to meet the practical necessities which govern a dam top.

The formula does not satisfy all the conditions, mathematically, for the dam is neither a complete cylinder with free ends, nor is it everywhere free to expand and contract; on the contrary, it is a segment connected at its bottom and sides with the immovable, but not the less elastic, country rock; and it is not a thin cylinder, but one of appreciable thickness. The problem is impossible of exact mathematical solution, for, however approached, its treatment is based on assumptions. The break of continuity in both form and material at the line of contact between the artificial structure and the natural rock, and the unknown value of the element of deformation in the natural rock, create uncertainties which, in themselves, render a mathematical demonstration based solely on absolute facts an impossibility, for the facts are not and cannot be known. Such elements of uncertainty are not confined to arch dams alone, they are equally prevalent with gravity dams. The researches and experiments of Pearson, Otley,

Brightmore, Baker, and others have shown the accepted middle-third method of gravity section design to fall short of being absolutely correct. Nevertheless, the fact remains that dams designed on that theory,

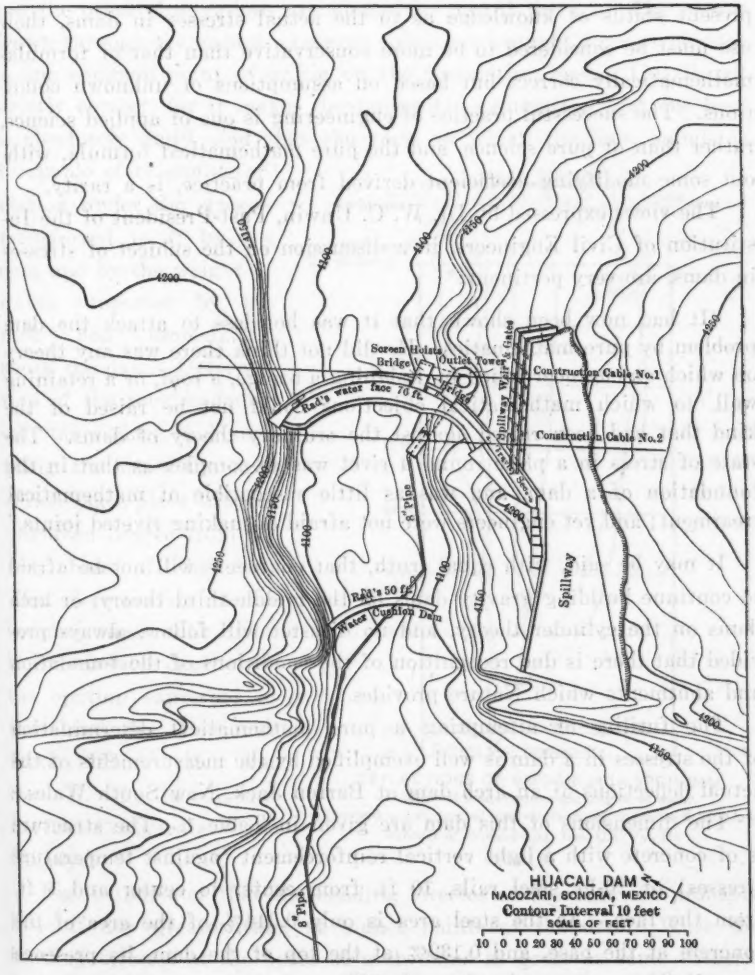


FIG. 5.

within certain recognized limitations as to intensity of stress, have been and continue to be successful. Similarly, the many Australian arch dams designed on the simple cylinder theory have been and continue to be successful.

Whatever the unknown and undeterminable stresses may be, particularly those around the foundations, the adopted formulas, empirical though they may be, have given and do give safe results, and, in the present status of knowledge as to the actual stresses in dams, their use must be considered to be more conservative than that of formulas mathematically correct but based on assumptions of unknown conditions. The successful practice of engineering is one of applied science, rather than of pure science, and the pure mathematical formula, without some modifying coefficient derived from practice, is a rarity.

The views expressed by Dr. W. C. Unwin, Past-President of the Institution of Civil Engineers, in a discussion on the subject of stresses in dams, are very pertinent:*

"It had now been shown that it was hopeless to attack the dam problem by pure mathematics. He did not think there was any theory on which an engineer relied in designing a bridge, a roof, or a retaining wall, to which mathematical objections could not be raised of the kind that had been raised against the ordinary theory of dams. The state of stress in a plate round a rivet was as complex as that in the foundation of a dam, and was as little susceptible of mathematical treatment; and yet engineers were not afraid of making riveted joints."

It may be said, with equal truth, that engineers will not be afraid to continue building gravity dams on the middle-third theory, or arch dams on the cylinder theory, and no disaster will follow, always provided that there is due recognition of the conditions of the foundation and abutments which Nature provides.

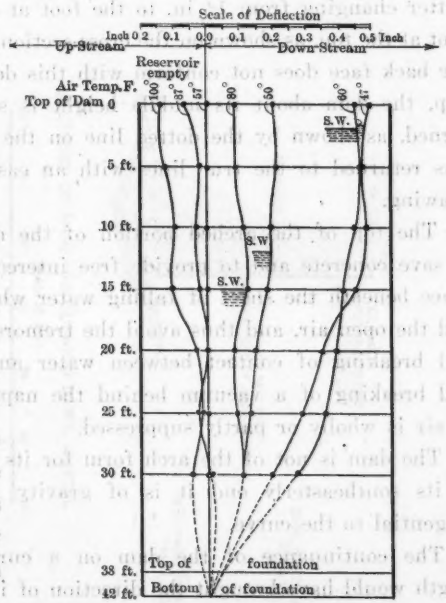
The futility of attempting a pure mathematical determination of the stresses in a dam is well exemplified by the measurements of the actual deflections of an arch dam at Barren Jack, New South Wales.†

The dimensions of this dam are given in Table 1. The structure is of concrete with a light vertical reinforcement (against temperature stresses) of 20-lb. steel rails, 10 ft. from center to center and 1 ft. from the face. As the steel area is only 0.044% of the area of the concrete at the base, and 0.139% at the top of the dam, its presence is negligible, as far as bending stresses are concerned.

The deflections of the Barren Jack Dam, which are shown in the diagram, Fig. 6, were taken under varying conditions of tempera-

* *Minutes of Proceedings*, Inst. C. E., Vol. 172, p. 156.
† *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 61.

ture and water depth. The observations made when the reservoir was empty and the air temperature 57°, which approximates the closest of the observations to mean temperature, are drawn as a straight line, and the other observations are plotted in regard thereto. The curves drawn through the various observed points are brought for convenience to one common point of origin on the foundation base. This is not strictly correct, for it makes the impossible assumption that the base is absolutely rigid, and that the rock on which the dam stands is incapable of elastic deformation under the stresses transferred to it by the dam and by the weight of water supported by the rock bed immediately above the dam. Deformation of the rock undoubtedly takes place, and, such being the case, the arch action takes place in some unknown degree from the very base of the dam. That deformation of the bed-rock does take place to considerable depths was the opinion expressed by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in discussing the Coolgardie Water Supply.*



DEFLECTIONS OF BARREN JACK ARCH DAM
At different temperatures and heights of water.
Measured at 5-ft. vertical intervals from top of dam to 30-ft. below
Minutes of Proceedings, Inst. C. E. Vol. 178

FIG. 6.

Fig. 6 indicates that the bending stresses at the base are probably little if any greater than at higher points of the dam. Were the bending stresses at the bases of arch dams as great as some theories would lead us to think, many if not all the arch dams cited would have been fractured, and in particular the very thin dams of Bear Valley and the Upper Otay. That they have not been fractured is conclusive that the theories need amending.

* *Minutes of Proceedings*, Inst. C. E., Vol. 162, p. 123.

The extreme height of the Huacal Dam from base to crest is 100.25 ft. The foundation trench was excavated until sound rock was reached at depths from 5 to 15 ft. That there might be no powder cracks, hand tools only were used in the bottom of the trench. Similar precaution was exercised in the abutment trenches at each end of the dam, which were carried horizontally into the rock formation to a maximum distance of 20 ft. to insure contact with sound rock.

The front face of the dam is vertical, the back face has a variable batter changing from 1¾ in. to the foot at the bottom, to ½ in. to the foot at the top, as shown on the cross-section, Fig. 7. As actually built, the back face does not conform with this design precisely. By a mishap, the dam about its middle height is somewhat thicker than designed, as shown by the dotted line on the cross-section. The profile was returned to the true lines with an easy batter, as shown in the drawing.

The top of the arched portion of the dam is made hollow, both to save concrete and to provide free intercommunication between the space beneath the sheet of falling water when the dam is overflowed, and the open air, and thus avoid the tremors incidental to the making and breaking of contact between water and dam with the forming and breaking of a vacuum behind the nappe when the free passage of air is wholly or partly suppressed.

The dam is not of the arch form for its entire length. For 42 ft. at its southeasterly end it is of gravity section, and in direction tangential to the curve.

The continuance of the dam on a curve throughout its entire length would have brought the direction of its easterly end too closely parallel to the axis of the canyon to have been economical or safe.

In the canyon below the main structure there is a supplementary dam designed to create at times of overflow a water cushion with a maximum depth of 20 ft. over the base of the main dam.

Spillway.

No stream discharge records being available, the determination of spillway capacity necessary for safety became largely speculative and matter of opinion. The subject was studied in conjunction with D. C. Henny, M. Am. Soc. C. E., and after weighing all available data it was concluded that provision to pass a flood of 6 000 sec-ft. would

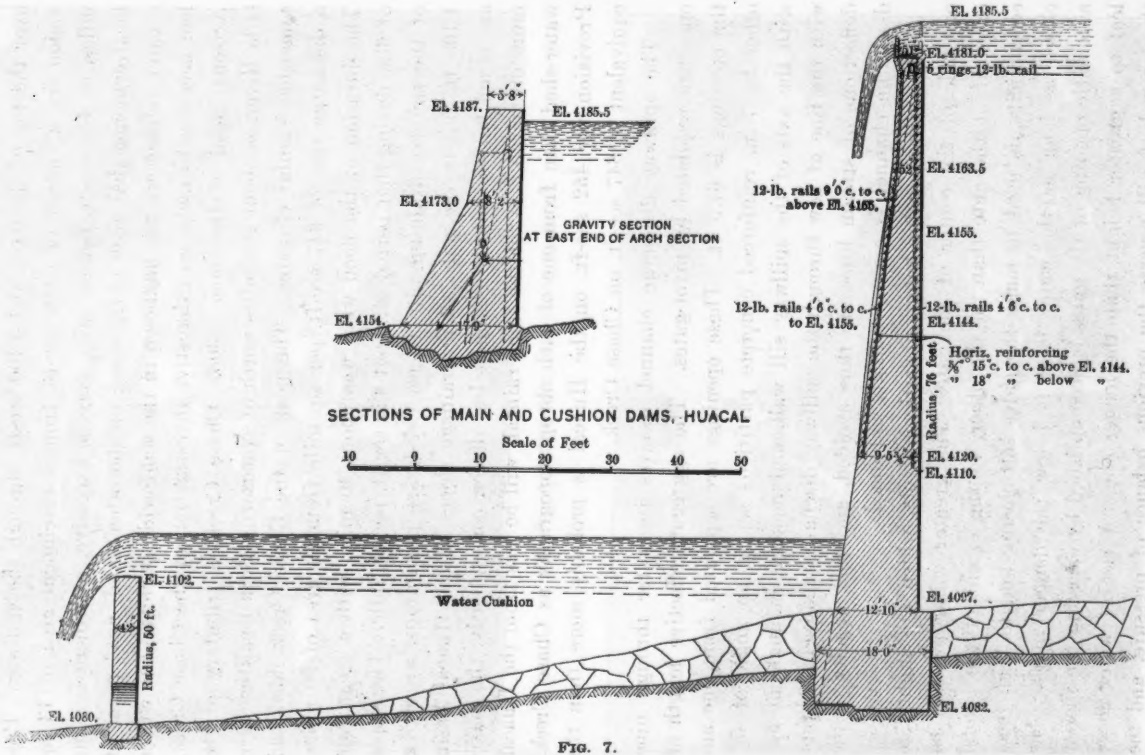

El. 4185.5
El. 4181.0
5 rings 12-lb. rail
El. 4187.
5'8"
El. 4185.5
El. 4173.0
8½
GRAVITY SECTION
AT EAST END OF ARCH SECTION
El. 4154.
11'0"
El. 4163.5
12-lb. rails 9'0" c. to c.
above El. 4165.
El. 4155.
12-lb. rails 4'6" c. to c.
to El. 4155.
12-lb. rails 4'6" c. to c.
El. 4144.
Horiz. reinforcing
⅜" 15" c. to c. above El. 4144.
" 18" " below "
Radius, 76 feet
9'5"
El. 4120.
El. 4110.
SECTIONS OF MAIN AND CUSHION DAMS, HUACAL
Scale of Feet
10 0 10 20 30 40 50
El. 1102.
Water Cushion
El. 4027.
12'10"
18'0"
El. 4082.
Radius, 50 ft.
El. 1050.

FIG. 7.

be ample. If this quantity is ever attained, 2 600 sec-ft. would flow over the top of the dam, and 3 400 sec-ft. through the spillway. Provision for a run-off of 6 000 sec-ft. from a catchment area of 13 sq. miles, or 452 sec-ft. per sq. mile, may be excessive, and there is some local evidence that such is the case; on the other hand, flood peaks of greater volume are not unknown in the semi-arid districts. The writer had occasion to make an exhaustive investigation of the Chase Creek, Arizona, flood of December, 1906. Chase Creek, a tributary of the Gila River, is about 200 miles north of Nacozari. Both localities have somewhat similar climates. The peak of the Chase Creek flood had a discharge rate of 647 sec-ft. per sq. mile from an area of 20 miles. The duration of the flood wave was about 40 min., with a mean rate of flow during that period of about 420 sec-ft. per sq. mile. The slopes of the country draining into Chase Creek are precipitous, averaging 1 100 ft. fall to the mile. The drainage slopes of the Huacal Basin are much flatter, and the channels less direct, consequently, the maximum discharge rate for the same rainfall will be less from the Huacal water-shed than from one of such abrupt topography as Chase Creek. Provision for 452 sec-ft. on the Huacal is probably more than the equivalent of 647 sec-ft. on Chase Creek.

The spillway discharge channel receives its water through nine openings regulated by pivot-gates. The aggregate effective length of the openings is 67.5 ft. These openings are separated from one another by 7-in. reinforced concrete partitions set at an angle of 45° with the axis of the spillway sill wall, and making an angle of 27° with the line of flow through the spillway discharge channel. Abrupt deflections of stream lines are thus avoided, and the capacity of the discharge channel is correspondingly enhanced.

The spillway gates hang on vertical pivots, which are slightly to one side of the center of the gate. The greater water pressure on one leaf of the gate than on the other, to the extent of the pivotal eccentricity, is used to make the gates self-closing and to keep them closed as long as the water level is below the danger mark. The automatic opening of the gates is accomplished by carrying the shorter leaf to a greater height than the longer one. A rise of water above the lower leaf will reverse the previous balance of pressure, and the gates will open automatically.

FIG. 8.—A QUIET DAY. LABORERS SCARCE.

FIG. 9.—A BUSY DAY.

Outlet System.

The discharge of water from the reservoir is effected by pipes controlled by flap-valves outside and standard screw-valves inside a concrete outlet tower, Fig. 10. The intake ends of the pipes are protected against trash by screens movable for cleaning.

The short bridge connecting the top of the tower with the dam is not connected to it rigidly. This leaves the tower free for independent movement in the event of an earthquake.

The gate-tower is a continuation of a shaft in the rock formation, from the bottom of which a pipe tunnel, in rock, runs out under the dam. The pipes consist of an 8-in. service pipe, a 12-in. blow-off, and a 10-in. drain. The tunnel is refilled with concrete in which the 12-in. and 10-in. pipes are embedded, together with an additional 10-in. pipe through which runs the 8-in. service pipe. This arrangement makes the service pipe removable for repairs or renewal. Should the other pipes rust out, the holes left through the concrete will answer all purposes.

The lower 8-in. intake pipe and the 12-in. blow-off are carried through a short tunnel into the tower-shaft. These pipes are of $\frac{3}{4}$-in. cast iron, embedded in the concrete of the tower. All these pipes are provided with the usual collars or flanges to stop seepage.

Cement and Sand.

The cement used was furnished, under the Standard Specifications of the American Society of Civil Engineers for Portland Cement, by the Southwestern Portland Cement Company, El Paso, Tex.

Tests at the dam, of neat cement, and cement and sand briquettes, gave the minima shown in Table 3.

The appearance and feel of the Huacal sand were not reassuring, but as the use of the local sand, if possible, meant a material reduction in cost, and as the cement and sand briquette tests promised well, a trial of the Huacal sand was made in a block of concrete containing some 80 cu. yd. The results were anything but satisfactory. After 10 days of setting a pick could be driven 1 in. into the mass without any great muscular energy. The total mass was removed, and the hope of using local sand was abandoned.

Good sand is always more or less scarce in arid regions, a fact largely attributable to insufficient natural grinding and washing to destroy and remove the softer particles. This is particularly true of locations such

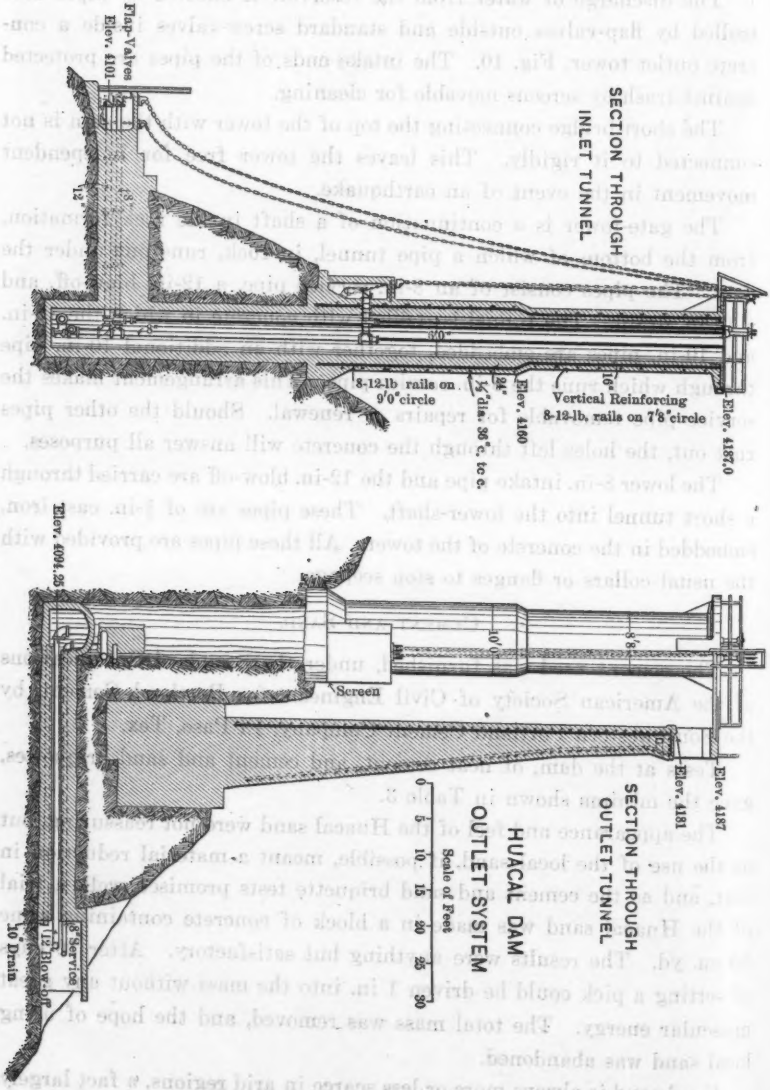

SECTION THROUGH INLET TUNNEL
Flap-Valves
Elev. 4101
8'
12"
8'
8'
8'
80'
8-12-lb rails on 9'0" circle
24"
½" dia. 36" c. to c.
Elev. 4160
18"
Vertical Reinforcing
8-12-lb. rails on 7'8" circle
Elev. 4187.0
HUACAL DAM
OUTLET SYSTEM
Scale of Feet
0 5 10 15 20 25 30
SECTION THROUGH OUTLET TUNNEL
Elev. 4094.35
Screen
10'0"
8'8"
Elev. 4181
Elev. 4187
8" Service
1½" Blow-off
10' Drain

Fig. 10.

as the one in question, where the greatest length from the dam site to the basin crest is 5 miles, a distance far too short to expose the rock detritus, in its journey along the various stream beds, to the scouring and abrading influences requisite to making good sand, however suitable the parent rock might be.

TABLE 3.—TENSILE STRENGTH OF CEMENT, IN POUNDS PER SQUARE INCH.

NEAT CEMENT BRIQUETTES.

Age of briquette, in days	1	3	4	5	6
Cement taken at random from store	290	485	685	660	640
Cement taken at random from mixing platforms.	210	.....	.....	675	790
Initial set: 2 hours 10 min. to 2 hours 30 min.					

1 CEMENT TO 3 SAND BRIQUETTES.

Description of Sand :					
Standard test	.....	215	.....	300	310
Nacozari, unwashed	.....	.....	88	100	.....
Huacal, unwashed	.....	180	.....	130	235
Nacozari, washed	.....	.....	.....	150	.....
Huacal, washed	.....	95	.....	155	165
Huacal, washed (on 200-screen)	.....	.....	160	190	150
Crusher-run screenings, ¼-in. mesh	.....	.....	155	.....	.....
Mixed Sands:					
3 Nacozari, 1 Huacal, washed	.....	.....	.....	100	.....
1 " 1 " unwashed	.....	.....	106	.....	.....
4 " 1 " "	.....	.....	75	.....	.....
1 " unwashed and 1 crusher-run screenings, ¼-in. mesh	.....	.....	125	.....	.....

CONCRETE.

The writer's previous experience having shown a 1 to 2 mortar to be safe for water-tight concrete, and that a 1 to 2½ mixture bordered on the pervious, a proportion of 1 of cement to 2¼ of sand was adopted, giving an excess of about 25% of cement over the voids in the sand. The coarse aggregate was crusher run, graded from fragments of ¾ to 1 in. down to a coarse, sharp sand. The average of numerous tests gave the void spaces of the crusher-run aggregate, after compaction by shaking and jarring, as 38 per cent.

A series of tests was conducted to determine the proportion of sand to crusher run which would give a mixture of greatest density. These tests were made by progressive weighing of a box of known volume filled with the mix to be tested, the box being shaken and jarred during the process of filling. The greatest average weight was attained with 1 of sand to 2.03 of crusher run, the void space being 19 per cent.

The proportions actually fed to the mixers averaged 1 cement: 2 sand: 3.85 crusher run. The sand of the crusher run added to the

straight sand made the desired 1 to 2¼ mortar. The proportion of sand in the crusher run varied, of course, from time to time, with the wear and changing and consequent slacking and tightening of the crusher jaws. The foreman watched the composition of the crusher run closely, and from his observations adjusted from time to time the sand component so as to maintain practically constant the desired ratios.

TABLE 4.—WEIGHTS OF SAND AND ROCK.

Kind of sand.	Specific gravity.	Pounds per cubic foot.
Nacozari, slightly damp, loose	....	105.2
Huacal " " "	....	101.8
Crusher-run, loose	....	104.8
Nacozari	2.51	157.0
Huacal	2.43	152.0
Andesitic rock fed to crusher	2.65	166.0

TABLE 5.—SCREEN ANALYSIS OF SAND.

Screen No.	Percentage Retained.		Percentage Passed.	
	Nacozari.	Huacal.	Nacozari.	Huacal.
20	71	60	29	40
30	12	21	17	19
40	5	8	12	11
60	5	4	7	7
80	2	1	5	6
100	1	2	4	4

The resultant mix gave a mortar approximately from 35 to 40% in excess of the voids in the coarse aggregate. Density of concrete was obtained with a sufficient surplus of mortar to coat thoroughly the surfaces of the 20-lb. to 300-lb. "plum" stones which were introduced as liberally as practicable. The "plums", first well wetted, were dropped into the mush concrete and worked around to insure liberation of any imprisoned air. The "plums" constitute probably 15% of the entire mass of concrete; a larger percentage would have been added if the hoisting facilities had permitted. The consumption of cement, as a whole, averaged about 1.3 bbl. per cu. yd. of completed concrete. Samples of finished concrete broken out of the actual structure weighed, dry, 152 lb. per cu. ft.

The concrete has proved to be water-tight to a marked degree, such slight dampness as appears in spots on the back face of the dam

being more probably due to seepage at over-night joints between one day's work and another than to percolation through the body of the concrete.

Plant.

The concrete plant was erected on the east side of the canyon at a higher elevation than the top of the dam. Cement was unloaded directly from the pack animals into store at the highest point of the plant, and the transportation was paid for by count of the sacks delivered. Sand, with transportation paid for by delivered weight, was unloaded and weighed into a bin near the bottom of the canyon and hoisted by bucket on inclined cable and delivered into gravity bins. The rock crusher was operated at an elevation corresponding with the spillway floor, and fed by rock from the spillway excavation. The product of the crusher was raised by belt and bucket elevator to the bins.

Cement, sand, and crushed rock gravitated from their respective bins to the batch-measuring hoppers, and thence to two mixers, one with a capacity of 12.7 cu. ft. to the batch, and the other of 21.7 cu. ft. The average composition of the mix was 1 cement: 2 sand: 3.85 crusher run.

The mixers delivered into a 1½-cu. yd. bucket slung between two standing cables, one crossing the canyon immediately below the dam and the other immediately above. Traveling carriages, one on each cable, moved in and out together. Hoisting tackle, independently operated, reached from each traveling carriage to the bucket bail. This arrangement permitted the depositing at will of the bucket at any point on the dam. Traveling and hoisting ropes were operated at speeds of about 600 ft. per min. light and 400 ft. per min. loaded. The maximum quantity of concrete placed in any one day was 114 cu. yd., and the average per day for the days on which the plant was running was 37.5 cu. yd. The minimum working time of mixing and placing 1 cu. yd. of concrete was 3.2 min., the maximum was 19.25 min. per cu. yd. (refilling outlet tunnels), with an average working time of 8.7 min. per cu. yd. for the whole work.

The machinery, picked up around the company's mines, and not without some incursions into the scrap pile, was of a heterogeneous character, incompatible with compactness or efficiency of operation. A saving in plant investment was effected at the expense of the constructional cost of the dam.

There were five steam boilers, ranging from 10 to 35 h. p. supplied with feed-water by a direct-acting, duplex, steam pump, drawing from water impounded above the dam. Six engines were used, a 25-h. p. for hoisting and a 30-h. p. for traversing the carriages on the cableways, a 26-h. p. on the rock crusher, a 30-h. p. on the sand hoist, and two 6-h. p. engines driving the concrete mixers. The fuel supply was chiefly wood from the live oaks cleared from the reservoir, liberally helped out with coal to compensate for the wood being green.

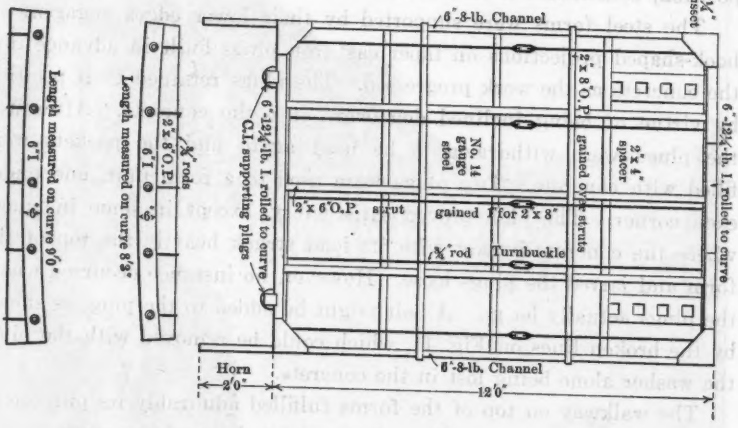
It would have been feasible to operate by electric current transmitted from the copper company's generating plant at Nacozari. This method was contemplated originally, but at the time was not deemed best. The experience of the work showed that the elimination of boiler and enginemen troubles and the elimination of transportation of coal, and other economies which would have followed with electricity as the motive power, would have resulted in a lower cost of construction.

The work was carried on during the Madero Revolution, when the country was in a state of turmoil; near-by collisions between the opposing forces were frequent, and the labor supply was disturbed and uncertain.

Forms.

The passing of any bolts, wire, or other form fastening, through from face to face of the dam was prohibited. The lower portion of the dam was constructed with wood forms of the customary stud and sheeting type, to a height above which the external struts bracing the studding would have become inconveniently long. The upper 60 ft. of the dam was built with steel forms (Fig. 11). These were in units 9 ft. wide for the water face and 8 ft. 3 in. for the back face, and 12 ft. high. They were generally used in threes, making a block space for pouring concrete 27 ft. by 12 ft. and of the width of the dam.

No attempt was made to vary the radius of the back face forms to conform to increasing radius due to decreasing thickness of dam with height. The forms were shaped to a compromise radius. The slight angle made by the adjacent forms, where not strictly conforming to the true radius, was imperceptible on the finished work. The location of the dam and its commercial use did not warrant the increased cost of forms of variable radius which would have been necessary for precision of shape. The front face of the dam being vertical,


Length measured on curve 9'0"
6'1"
Length measured on curve 8'3"
6'1"
2"x 8" O.P.
¾" rods
6"-8-lb. Channel
¼" gusset
6"-12¼-lb. I, rolled to curve
6"-12¼-lb. I, rolled to curve
C.I. supporting plugs
2"x 8" O.P. grained over struts
2"x 4" ejector
No. 14 gauge steel
2"x 6" O.P. strut
gained 1" for 2"x 8"
¾" rod
Turnbuckle
Horn
2'0"
6"-8-lb. Channel
12'0"

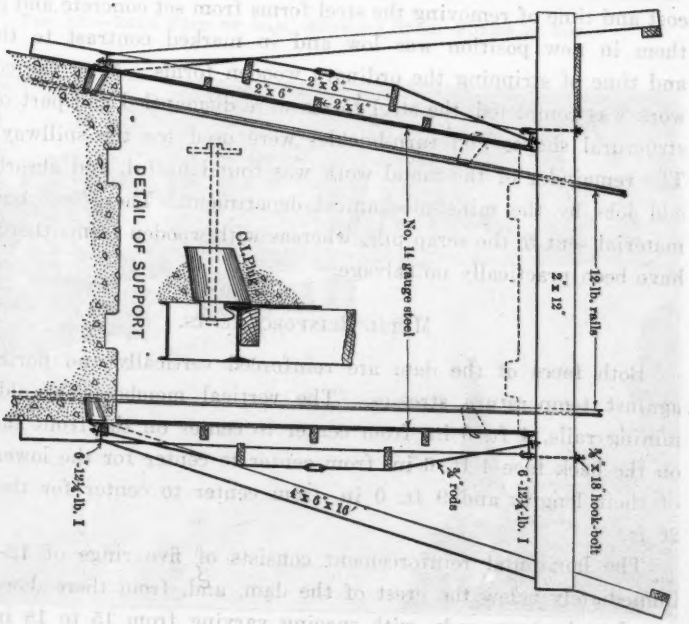

2"x 8"
2"x 6"
2"x 4"
No. 14 gauge steel
2"x 12"
12-lb. rails
C.I. Plug
DETAIL OF SUPPORT
¾" rods
¾"x 18" hook-bolt
6"-12¼-lb. I
4"x 6"x 10"
6"-12¼-lb. I
STEEL FORMS FOR HUACAL DAM
SCALE OF FEET
0 1 2 3 4 5

FIG. 11.

its radius is constant. The front forms, being first set accurately to position, constituted a base from which were set the back forms.

The steel forms were supported by their lower edges engaging in hook-shaped projections on taper cast-iron plugs built in advance into the concrete as the work progressed. The plugs retained their position by virtue of being inclined downward into the concrete. After use, the plugs were withdrawn to be used again and the pockets were filled with concrete. Two plugs were used to a form unit, one under each corner. The plan proved satisfactory, except in some instances where the concrete bucket with its load struck heavily the top of the form and jarred the plugs loose. However, no instance occurred where the plugs actually let go. A bolt might be added to the plug, as shown by the broken lines on Fig. 11, which could be removed with the plug, the washer alone being lost in the concrete.

The walkway on top of the forms fulfilled admirably its purpose of a safe and convenient working platform for the concrete gang.

The forms were handled into place by the overhead cableways. The cost and time of removing the steel forms from set concrete and placing them in new position was low and in marked contrast to the cost and time of stripping the ordinary wooden forms. After the concrete work was completed, the steel forms were dismantled and part of their structural shapes and turn-buckles were used for the spillway gates. The remainder of the metal work was found useful, and absorbed for odd jobs by the mine mechanical department. There was but little material sent to the scrap pile, whereas with wooden forms there would have been practically no salvage.

Metal Reinforcements.

Both faces of the dam are reinforced vertically and horizontally against temperature stresses. The vertical members are old 12-lb. mining rails, 4 ft. 6 in. from center to center on the front face, and on the back face 4 ft. 6 in. from center to center for the lower 35 ft. of their length, and 9 ft. 0 in. from center to center for the upper 26 ft.

The horizontal reinforcement consists of five rings of 12-lb. rail immediately below the crest of the dam, and, from there down, $\frac{5}{8}$-in. round and square rods, with spacing varying from 15 to 18 in. from center to center.

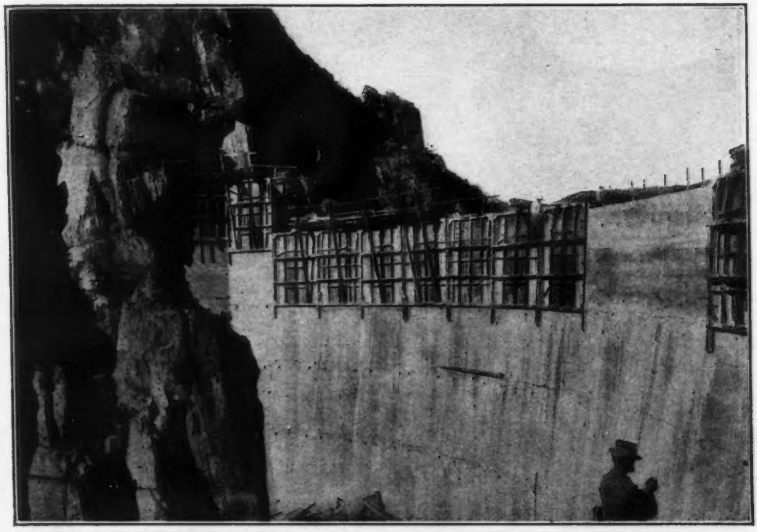


FIG. 12.—REAR FACE OF DAM, SHOWING STEEL FORMS IN USE.

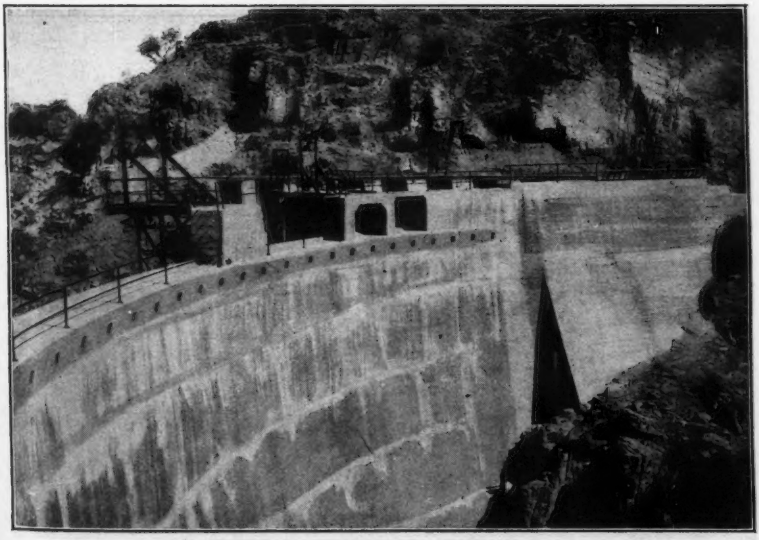
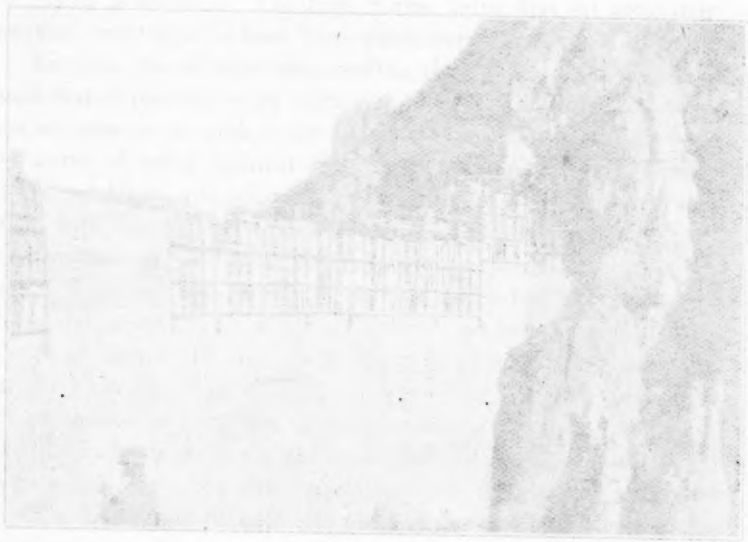
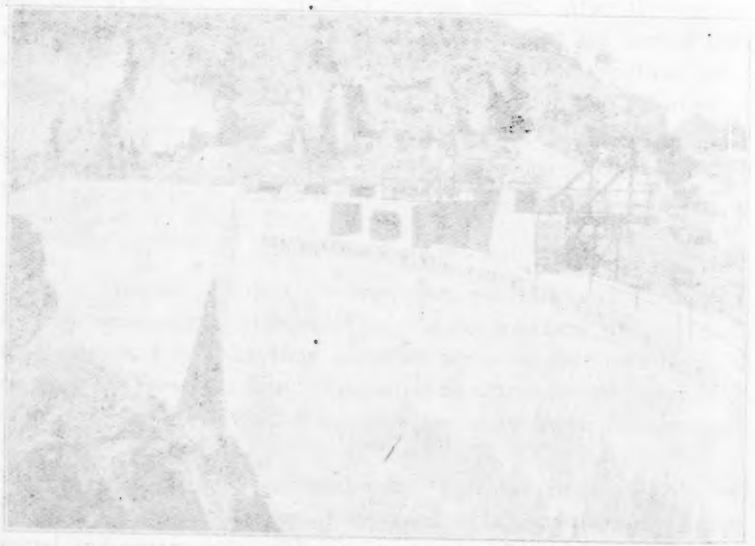


FIG. 13.—UPPER PORTION OF DAM, REAR FACE. GRAVITY SECTION ABUTMENT AT EAST END.





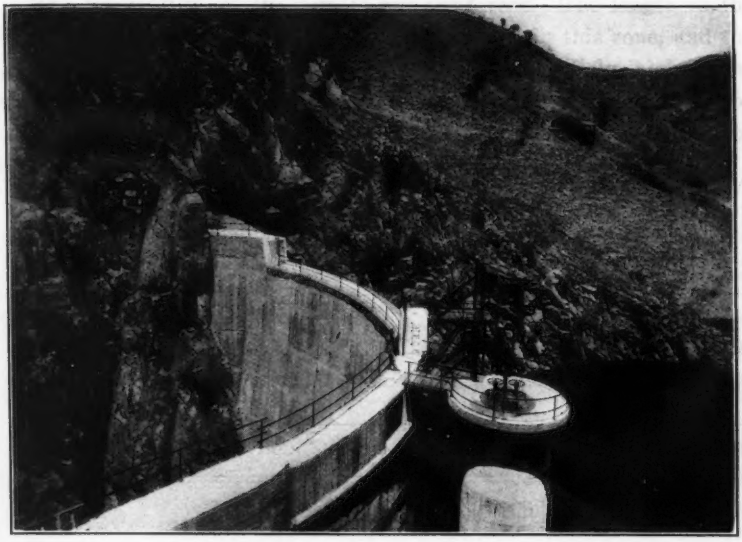
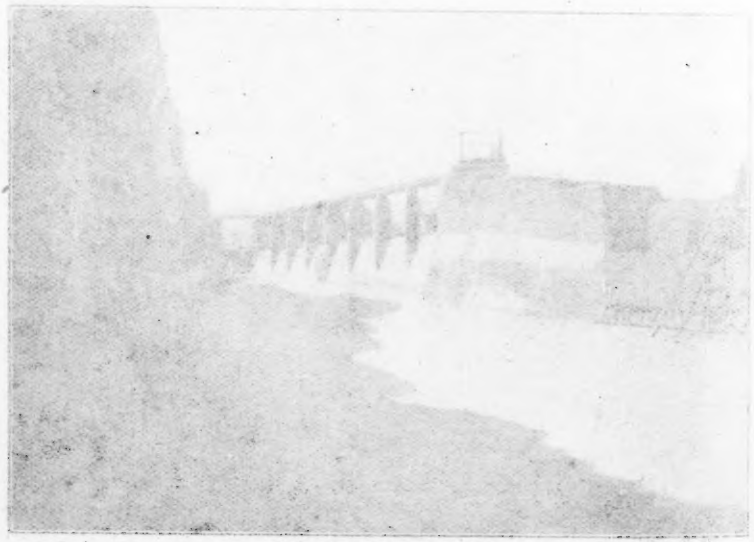


FIG. 14.—TOP OF DAM, FROM SPILLWAY.

FIG. 15.—SPILLWAY. GATES NOT IN PLACE.



The lower 30 ft. of the dam is not reinforced. The length of the dam, measured on the arc, is comparatively small in this zone, and the possible temperature range is materially less than in the higher portions. Practically, the front face of the dam for this depth will be constantly immersed, which tends to uniform temperature. On the back face the lower part is less exposed to the sun than the upper.

TABLE 6.—CONCRETE QUANTITIES.

		Cubic yards.
Main dam and outlet tower		4 093
Spillway weir	475 cu. yd.	
Spillway floor and retaining wall	308 " "	783
Water-cushion dam		82
Miscellaneous		30
Total		4 988

PIPE LINES.

Water from the reservoir is conducted to the mill tanks at Nacozari through a pipe line, 2.91 miles long, composed of 1.13 miles of 8-in. and 1.78 miles of 10-in. steel riveted pipe, both of No. 14 gauge, furnished by the Lacy Manufacturing Company, of Los Angeles. Only on the lower 3 600 ft. of the line could wagons be used for the distribution of the pipe; for the remaining distance the pipe had to be packed by animals and men. Under these conditions, the savings to be made by using light riveted pipe outweighed considerations of permanency.

For wagon transportation, the pipe was made up in 20-ft. sections, and for pack animals in 10-ft. sections. Where buried in the ground, the usual California drive joints were used, and where exposed, flanged joints. The flanges are of pressed steel, riveted to the pipe. The flange dimensions were: for 10-in. pipe, 14¼ in. in diameter, 12¾-in. bolt circle, 12 ½-in. bolts; and, for 8-in. pipe, 12¼ in. in diameter, 10¾-in. bolt circle, 10 ½-in. bolts. When pipe had to be cut, or articulation was needed, bolted joints of the loose-ring type, with triangular rubber gaskets, were used. For water-tightness, the drive joints are dependent on the asphalt coating which the pipes receive in the final process of manufacture. In the field, after entering the end of the driven pipe, kerosene is poured around the outside of the joint and fired, and if necessary refired, until the asphalt coating softens and runs; then the

TABLE 7.—Cost of Dam.

				Per cubic yard of concrete.	
Roads and trails, labor			$1 528	$0.306	
Plant:					
Buildings, material	$2 985				
" labor	803	$3 788			
Sand bins, material	$970				
" labor	416	1 386			
Cableways, material	$1 574				
" labor	734	2 308			
Machinery and supplies, material	$4 363				
Installing machinery, labor	3 982				
Transporting machinery	1 190	9 535			
Water tanks		158	17 175	3.443	
Miscellaneous, unsegregated, material	$2 395				
" " labor	7 869		10 264	2.058	
Gates and control fittings, tower	$800				
Installing piping, tower, labor	880		1 680	0.337	
			$30 647		$6.144
Excavations:					
Outlet tunnel	$365				
Inlet tunnel	231				
Intake shaft	275	$871			
Spillway		2 264			
Foundation trenches, etc.		2 173	5 308		1.064
			$35 955		$7.208
Concrete:					
Cement at Nacozari	$26 840				
Transportation of cement	2 847	$29 687		$5.952	
Sand, screening and handling	$3 007				
Transportation of sand	8 430	11 437		2.293	
Coarse aggregate, crushed rock from spillway excavation. Cost of crushing not segregated.					
Forms:					
Lumber	$4 712				
Steel	2 600				
Labor	6 151				
Transportation	1 189	14 652		2.937	
Mixing and placing concrete, and crushing rock, labor	$6 541				
Fuel for above, coal	$1 318				
Transportation of coal	267				
Wood	2 048	3 633	10 174	2.040	
Blacksmithing, labor	$1 248				
" material	408	1 656		0.332	
Reinforcing material		1 454	69 060	0.291	$13.845
			$105 015		$21.053
Superintendence			2 790		0.559
Engineering, field		$2 815		$0.564	
" plans and supervision		3 670	6 485	0.736	1.300
Total			$114 290		$22.912

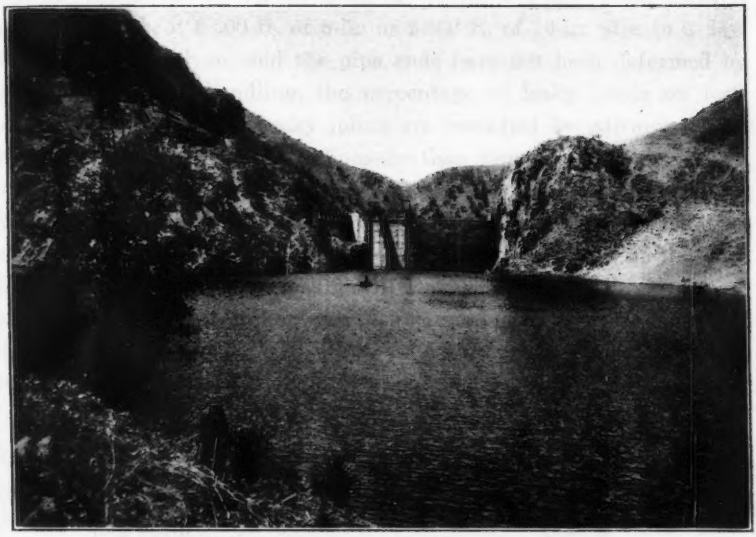


FIG. 16.—FRONT OF DAM, WITH 53 FEET OF WATER.

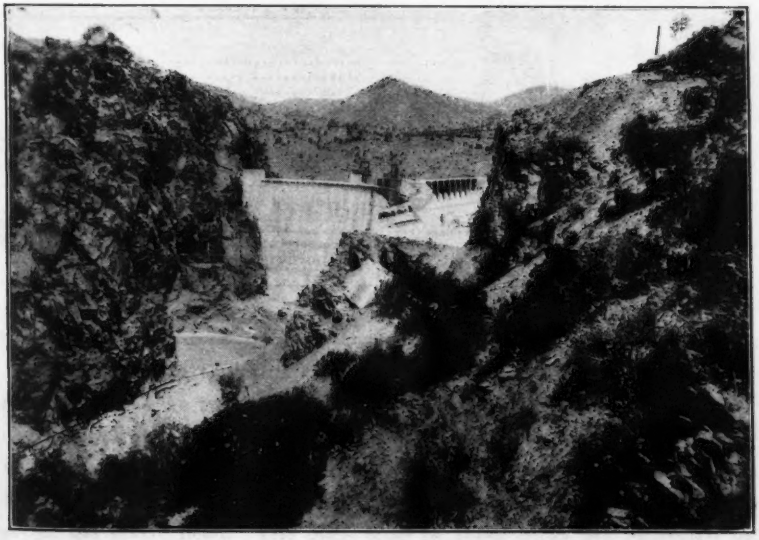


FIG. 17.—MAIN DAM, CUSHION DAM, AND SPILLWAY.

pipe is driven home. In straightaway work, a gang of experienced men can lay about 2 500 ft. of 8-in. or 2 200 ft. of 10-in. pipe in a day. When properly driven, and the pipe ends have not been deformed by transportation and handling, the percentage of leaky joints on testing out is very small. Leaky joints are remedied by galvanized-iron sleeves of sufficiently greater diameter than the pipe to leave an annular space of about $\frac{3}{4}$ in. into which neat cement or cement mixed with fine sand is rammed. When, from necessity, sleeves have to be put on a pipe under pressure, a closeable vent should be provided at the mid-length of the sleeve to discharge all leakage water until the cement sets. Seam leaks of minor nature are curable by floating fine sawdust through the pipe. With large seam leaks, it is preferable to put in a new section of pipe.

TABLE 8.—COST OF PIPE LINE.

Huacal pipe line		5 974 ft. 8-in. pipe.	
		9 415 " 10- " "	
Total length		15 389 ft.	
		Total cost.	Average cost per linear foot.
Material:			
Pipe and fittings	$12 456		
Trestles	1 857		
Grading tools and supplies	561		
Transportation of material	790	$15 664	$1.02
Labor:			
Grading	$4 546		
Framing and erecting trestles	1 051		
Laying and covering pipe	2 056	7 653	0.49
		$23 317	$1.51
Superintendence		310	0.03
Engineering, field	$950		
" designing and supervision	816	1 766	0.11
Total		$25 393	$1.65

Break-pressure tanks of concrete, with "Davis" float-valve control, are introduced at two convenient points on the line, which limit the maximum static pressure to 179 ft. for the 8-in. and 283 ft. for the 10-in. pipe. At the summits, air relief and prevention of vacuum are afforded by 2-in. pipes carried up the mountain side to an elevation above the static head. The open ends of these pipes are protected by wire gauze to prevent the intrusion of foreign substances and living

things. Where the situation did not permit of open pipes, "Simplex" combined air relief and vacuum valves were used. The breakage of the pipe or the sudden opening of a blow-off valve at a low point in the line, thereby causing a partial vacuum, are to be particularly guarded against in light pipes. A vacuum which might be without ill effect in a heavy metal pipe line is absolutely fatal to thin riveted pipes, which have but little strength to resist pressure from without.

Test of Pipe Capacity.

Color tests of the velocity of the water traversing the pipe line were made by using permanganate of potash. Through 4 354 ft. of the 8-in. line, with a fall of 170 ft., the velocity was found to be 7.223 ft. per sec., from which the discharge was computed as 2.521 sec-ft.; the corresponding value of n in Kutter's formula is 0.0114. The velocity through 9 415 ft. of the 10-in. pipe, with a fall of 110 ft., was found to be 4.689 ft. per sec., and the computed discharge 2.555 sec-ft.; and the value of n is 0.0116. A volumetric test, made at the mill tank, gave the discharge as 2.529 sec-ft. In view of the fact that the pipes and tank were not calibrated, the three determinations of 2.521, 2.555, and 2.529 cu. ft. per sec. may be said to be in perfect agreement, the difference between the highest and lowest determination being but 1.33 per cent.

The writer takes this occasion to express his thanks to Mr. J. S. Williams, Jr., General Manager, and to other officers of the Moctezuma Copper Company, for their aid and co-operation in the work, and to Mr. C. A. Dodge for his effective supervision of the work of construction.

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PAPERS AND DISCUSSIONS

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REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

BY HARRISON S. TAFT, ESQ.

TO BE PRESENTED MAY 20TH, 1914.

SYNOPSIS.

During the past few years considerable discussion has taken place among engineers and chemists as to the feasibility of using cement in sea water, and the practicability and commercial success of reinforced concrete docks. In the United States the construction of reinforced concrete docks is in a very embryonic state, and the concrete of structures in sea water has not always been successful. On the other hand, concrete has been used successfully for more than 50 years in Europe for structures exposed to the action of salt water, and English engineers have been building reinforced concrete docks for about 20 years.

Part I of this paper contains a brief outline of the reinforced concrete dock problem as developed in Europe and Asia, the work in each country being examined. Part II reviews the art as carried out in the United States, both in sea water and fresh water. These two parts cover descriptions of reinforced concrete docks, as well as construction features of special interest.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Part III takes up the subject of failures in past uses of cement in sea water in America, points out the reasons therefor, and brings out the fundamental principles covering the successful use of concrete in structures exposed to salt water and frost action. A foot-note calls attention to the chemistry of salt water, cement, and the use of concrete in sea water, in order to bring out discussion on this phase of the problem.

Part IV goes briefly into the question of the initial cost of reinforced concrete docks, as well as the cost of annual repairs.

Part V is a review of opinions of foreign expert dock engineers on the use of concrete in sea water and on the practicability of reinforced concrete docks; there is also a short review of the use of concrete in foreign breakwaters, closing with a few remarks as to the reasons for the success of foreign engineers in using concrete in sea water.

Part VI contains the writer's conclusions, wherein he cites the relative positions of English and American engineers in respect to the reinforced concrete dock question, and a more general use of concrete. He also speaks of progressive engineers possessed with the determination and courage to develop new ideas and uses for concrete, with due consideration as to previous failures, while conservative engineers still cling to old-time, long-established practices.

In the closing paragraph the writer calls the engineer's attention to the necessity of special training and knowledge for the successful execution of concrete structures in salt water.

The object of the paper is to make known to the Engineering Profession the exact up-to-date situation with respect to the development of this problem, the world over, the feasibility of using cement in sea water, and the practicability of building reinforced concrete docks.

Introduction.

In undertaking a study of the reinforced concrete dock problem, the subject has so many phases as to require a number of chapters to cover fully all the various ramifications into which it leads. From a world point of view, reinforced concrete docks are not a novelty. On the other hand, their practicability and commercial economy have perhaps as yet to be proven, especially in America. It is true that there are many reinforced concrete docks in successful operation in foreign countries, the most extensive development being found in England. As

their size and number are steadily increasing, it would appear that this class of dock construction has proved its worth, and sooner or later will become the standard type where a structure of a permanent nature is desired.

As reinforced concrete docks are somewhat of a novelty in the United States, it is perhaps necessary for engineers to seek the experience of foreign countries in order to obtain the necessary data on which to base their conclusions as regards the feasibility or otherwise of this type. Thus it is eminently fitting to investigate first what has been done in various parts of the world in the way of building such docks and the success accompanying such undertakings, and then discuss other phases of the problem, as to what precautions, etc., are necessary to render them a commercial, practical, and engineering success.

Part I.—Reinforced Concrete Docks in Foreign Harbors.

England.

In building reinforced concrete docks and other concrete structures in sea or fresh water, it is only natural that a forested country should be the last to take up the development of such a material. Consequently, America has been far in the rear in regard to this question, as compared with what other and older countries have accomplished. In England, France, Germany, Italy, and other European countries, in Australia, and in Asia, as well as in certain South American countries, concrete sea-walls, breakwaters, dry docks, piers, trestles, coaling stations, etc., in salt water, have been in existence for years. In looking up past undertakings along these lines, the writer was amazed to learn of the large number of reinforced concrete docks that have been built in foreign countries and the great success that has apparently been attained with them.

Southampton.—One of the most noted developments in reinforced concrete construction, as applied to harbor and dock development, is that of the London and Southwestern Railway Terminals at Southampton, England. The first and most prominent reinforced concrete structure in connection with this terminal was a coal barge jetty, 300 ft. long and 20 ft. wide, built in 1904, on the Hennebique system of driven concrete piles. The piles are about 44 ft. long, standing in 29 ft. of water at high tide, the rise and fall of the tide being about 13 ft. Each pile carries a maximum load of 17 tons.

This structure carries a very heavy traveling coal-hoisting apparatus for unloading coal from large vessels docked on one side, into harbor barges or scows which lie on the other side. Thus the jetty is subjected to constant blows from both sides, in addition to the heavy vibration due to the traveling machinery it supports.

In speaking of this jetty, Mr. Francis Wentworth-Shields, Dock Engineer for the London and Southwestern Railroad, says:

"Though the impacts from the vessels and scows cause this whole jetty to sway, there seem to have been no signs at the end of the first 2½ years of its existence of any of the concrete peeling off."

During recent years this dock or jetty is said to have shown some signs of deterioration, due to the vibration of the heavy machinery traveling along it, the supposition being that it was built too light in the first place to absorb the heavy vibrations to which it has been subjected. Still, the dock is said to have considerable elasticity. In one instance it was in heavy collision with a steamer, two piles and the beams they carried being broken. The dock was effectively repaired, but perhaps with some difficulty, though it is claimed that the repairs were easily accomplished.

The same railroad company has built several other reinforced concrete pile docks at Southampton, designed to carry the same heavy deck loads as the coaling jetty, but without the heavy vibration to which this jetty is subjected. Though built on the same system of construction as the coal jetty, the latter docks have shown no signs of wear or deterioration. It is said, on the best of authority, that they have cost nothing to date for maintenance.

The largest of these is the extension, on the Itchen Front, of "The Empress Dock", a widening of the so-called "Old Extension Quay" by a reinforced concrete pile structure, 50 ft. wide and about 1 300 ft. long, parallel to and securely dovetailed into the old quay wall. This widening dock is built of complete concrete bents, along the tops of which are steel deck-beams or stringers, which in turn are covered with 4 in. of wood, a wooden block pavement being used for the wearing surface. The depth of water at low tide at the face of this structure is 35 ft., which, with a 13-ft. rise and fall of the tide, gives a depth of 48 ft. at high tide.

One of the finest cold storage and cattle stations in existence was built at Southampton in 1905 to accommodate the foreign cattle trade.

The landing stage or jetty of this station is a reinforced concrete structure, 200 ft. long and 38 ft. wide, connected with the main land by two runways, 142 ft. long and 15 ft. wide.

On the opposite side of Southampton Harbor, at Woolston, on the Itchen, there is a reinforced concrete landing dock, 136 ft. long and 100 ft. wide, built in 1899 on the Hennebique system. This was the pioneer of reinforced concrete dock construction in Southampton waters. Up to date, this dock has cost practically nothing for repairs, except for damages due to the fact that it was rammed or otherwise damaged by a large steamer; it is in excellent condition at present. The cost of making the repairs is said to have been very small.

Up to the present time, it appears that at least six reinforced concrete docks, jetties, or quays, have been built in Southampton waters.

In building one of the Southampton docks, it is stated that some of the concrete piles were sprung out of line in driving them. A prominent American engineer reports that he saw a number of piles from 1 to 2 ft. out of line, but that they showed no signs of cracks. It has been stated that, in handling Chenoweth concrete piles, up to a length of 61 ft., and 13 in. in diameter, they were rolled about like wooden piles, at times having quite a spring in them. Under this treatment they showed a remarkable degree of elasticity and no signs of cracking.

Bending of Piles.—In discussing the bending of concrete piles, it is of interest to note a series of tests made on a hollow telephone pole in Fulham, London, England, in 1911. The pole was 44½ ft. long, 17 by 17 in. at its base (outside dimensions), tapering to 8 by 8 in. at its head. The thickness of the shell was 2 in., making the inside dimensions of the pole 13 by 13 in. at the base and 4 by 4 in. at the top. The vertical reinforcement consisted of 248 $\frac{3}{16}$-in. rods of high-tension steel, the ultimate tensional stress being from 80 000 to 85 000 lb.; 56 rods were grouped at each corner, 6 rods being spaced evenly on each of the four sides of the pole. The area of the concrete at the base was 106 sq. in., and the area of the steel 6.85 sq. in., a ratio of 0.0445. In making the test, the pole was set in 5½ ft. of massed concrete, with the pulling rope attached to its upper end.

In Table 1 is recorded the pull, in pounds, the deflection, and the permanent set after the loads had been released.

After applying the 6 000 lb., the test was discontinued for 3 days, and then the load was slowly increased to 9 200 lb., which gave a de-

flection of 66 in. and a permanent set of 21 in., with "cracks on tension side and permanent cracks more numerous and pronounced." There was no sign of failure on the compression side of the pole.

TABLE 1.—Test of a Hollow Concrete Pole at Fulham, England.

Loads, in pounds.	Deflection, in inches.	Permanent set, in inches.
300	¾	None that could be observed.
500	2	
1 000	4¼	
1 250	5	
1 500	6¼	
1 750	8	
2 000	9	
2 250	10⅛	
2 500	12	¼
2 750	14	¼
3 000	14¾	⅝
3 550	19½	¾
3 100	16½	⅞
3 500	18¾	⅞
4 000	22	¾
4 500	25½	1⅛
5 000	29	1¾
Load gradually increased to 5 750	34	Pulling wire broke. Pole flew back, vibrated short time and came to rest in vertical position with no permanent set.
Load gradually worked up to 6 000	37½	2 in. immediately after release of load. After interval of 1½ hours, set was reduced to 11/16 in.

Add to the above loads 100 lb. for weight of wire and recording apparatus, etc.

Notes on Behavior of Pile.—At 3 350 lb., slight hair cracks appeared on tension side at about 5 ft. 6 in. above foundation.

At 3 600 lb., slight hair cracks appeared at regular intervals about 6 to 9 in. apart.

At 4 100 lb., slight shear cracks appeared, starting at 33 in. above foundation and extending vertically, at a distance of 2 in. from tension side, for some 3½ ft. in length.

At 4 600 lb., hair cracks occurred regularly, 3 or 4 in. apart on tension side.

At 5 100 lb., cracks on tension side were noticed to be traveling across the sides of the pole to within 6 in. of compression surface.

At 6 100 lb., the shear cracks were more pronounced. The hair cracks on tension side were about 1 in. apart, but no signs of failure appeared on compression side.

The load was again applied. When a deflection of 73 in. was obtained, signs of failure appeared for the first time on the compression side, and the pole failed, but no record of the amount of the pull was obtained. The final examination showed that the pile failed equally for its entire length on the shear and tension sides, but without any local weakness. The compression side failed from the base up for a distance of about 2 ft. The bending moment at the base of the pile was reported to be 4 863 936 in-lb., a most remarkable test, which seems to substantiate the experience quoted above.

Liverpool.—The Mersey Docks and Harbor Board appears to have made a very limited use of reinforced concrete in its latest port develop-

ments, in spite of the extensive tidal and graving docks of concrete constructed at Liverpool. As a matter of fact, most of its first so-called reinforced concrete docks were in reality semi-concrete structures, that is, wooden piles carrying a concrete deck system. The oldest of these structures, the Cattle Wharf, Prince's Stage, was built in 1899-1901 on greenheart piles, with the usual Hennebique system of concrete deck-beams and slab. In addition to the Cattle Wharf, there are two other dock-quays in Liverpool, built on the same system, designed for a load of 3 tons per sq. yd., with a test load of 4½ tons.

Another of Liverpool's reinforced concrete docks is the Prince's Dock, West Quay, completed in 1905. This structure was designed to carry a super-load of 3 tons per sq. yd., or about 66 tons per pile. The piles are of concrete, 16 in. square, spaced 15 ft. 9 in. from center to center, across the dock, and 12 ft. 6 in. from center to center, longitudinally, and resting on rock.

At the west end of Liverpool's dock system is the so-called Brocklebank Dock, 226 ft. long and 64 ft. wide, a reinforced structure built in 1908 on the Hennebique system, and designed to carry the same loading as the Prince's Dock. The test load applied was 9 tons per sq. yd. The piles of this dock are 20 in. square, and had to sustain a load of about 95 tons each, when the test load was applied.

During the first year of its existence there was considerable trouble with the Cattle Wharf and other semi-concrete structures, due to the permeability of the deck-slab part of the structure. The dampness from the salt water entered the permeable concrete and attacked the steel reinforcement on the under side of the slab, causing pieces of concrete to fall off. The defect was overcome by applying a heavy coating of cement to the under side. The upper side of the deck-slab, which is washed daily, shows no signs of deterioration. This emphasizes the fact that great care must be taken in placing concrete in structures standing over or in salt water; there must be the closest inspection during their construction. As a result of this trouble, it appears that the steel reinforcement must be kept farther from the surface of the concrete in salt water concrete structures than in ordinary work. Whereas, at first a minimum of 1½ in. was supposed to be sufficient, the engineers of the London and Southwestern Railway have found this to be insufficient, and are now specifying a minimum of 2 in. for their new structures. Another difficulty encountered was the nu-

merous joints in the Liverpool system of dock construction. This appears to have been overcome by cleaning out the joints and filling them with a rich cement grout, with such success that the docks are reported to be in very satisfactory condition at the present time.

Thames River, Etc.—Within the bounds of the "Port of London Authority", and at other places on the Thames, many reinforced concrete docks or quays have been constructed. One of the most prominent of these is the Thames Haven Jetty Head, near the mouth of the river. This structure was built in 1908, for the berthing of large vessels, and is 136 ft. long and 32 ft. wide. It is supported by 19 columns or piers, 5 ft. in diameter up to low-water mark but of oval shape, 5 by 2½ ft. above. Each column rests on two 15-in. concrete piles of octagonal section, 45 ft. long, with a penetration of from 18 to 20 ft. After these concrete piles had been driven to refusal, a temporary iron caisson was put down over them. The reinforcing hoops were then lowered into place around the two piles and the caisson was filled with concrete. The caisson was afterward removed, being built in half sections.

Farther up the Thames, 20 miles from its mouth, is the Purfleet coaling jetty, built in 1904, a reinforced concrete structure, 250 ft. long and 34 ft. wide, carrying two heavy traveling cranes. The concrete piles are 14 in. square, from 40 to 50 ft. long, with from 15 to 18 ft. of penetration. The deck-slab is 5 in. thick. The jetty is connected with the mainland by a reinforced concrete approach, over which cars are run to the jetty itself. On one occasion this jetty was rammed by an 8 000-ton steamer, eight piles and 20 ft. of the decking being damaged. That the damage was not more extensive is said to have been due to the firmness of the horizontal decking. In repairing the dock it was necessary to withdraw the eight broken piles and drive new ones. In conducting this repair work, it was especially noticed that the steel reinforcing bars of the original structure showed no signs of rust, though it is well to note that the water at this point of the Thames may be brackish, and not real sea water. The repairs were efficiently made, and the quay is still doing duty.

Another interesting reinforced concrete coaling dock is at Dagenham-Essex-on-the-Thames, 10 miles below London, built in 1901. It is 780 ft. long and 35 ft. wide, and carries eight heavy traveling cranes, weighing 60 tons each, besides a railroad track. Each supporting col-

umn consists of three concrete piles encased in a concrete cylinder filled with concrete. Though it may seem a trifle odd, a careful study of photographs of this structure indicates that the tops of the columns are made with "capitals"—a well-proportioned and artistic structure as regards concrete dock work. A similar structure, known as the Prince of Wales Pier, was built in 1903 at Falmouth.

The Thames Iron Works and Shipbuilding Company has constructed at Dagenham a very substantial reinforced concrete dock, designed to carry a concentrated load of 60 tons, for the berthing and fitting out of dreadnaught battleships. In the construction of this dock, the Williams type of concrete piles was used for the first time. This type consists of an I-beam surrounded with $\frac{3}{16}$-in. wire, 12 in. from center to center, the whole being encased in concrete, with special provision at each end to take up the reaction of the driving.

In the same vicinity is the Hornchunk Dock and approaches, built in 1906. The main structure is 400 ft. long and 24 ft. wide, the approach being 280 ft. long and 16 ft. wide; both were built on the Hennebique system. The concrete pile bents are 14 ft. from center to center, braced diagonally and support a concrete beam and deck-slab system. Along the water-front side of this dock and its approach, a concrete curtain-wall was built between the piles. This curtain-wall in turn is protected by a wooden fender system. The plans seem to indicate that this structure was designed to carry a 27-ton crane in addition to a 27-ton locomotive.

A reinforced concrete coal dock was built in 1906 at Rochester, on "The Medway", a river flowing into the estuary of the Thames. This dock consists of a main water-front section, 32½ ft. wide and 340 ft. long, connected with the land by two concrete approaches, 100 ft. and 180 ft. in length, respectively. It was built on the Hennebique system, and carries a heavy traveling crane in addition to two railroad tracks. This was one of the first large undertakings in reinforced concrete dock construction on the Thames or its tributaries. At the time it was built it was looked on as the most important structure of its kind in England.

Though the exact number of reinforced concrete docks now in existence on the Thames or its tributaries is perhaps unattainable, it appears that they are more numerous in those waters than in any other part of England.

General.—Perhaps the largest reinforced concrete dock constructed up to date in England is at Swansea, on the southwest coast of Wales. It is a Hennebique design, almost 2 000 ft. long, completed in 1908, the whole structure being built in the dry. Some of its columns are 3½ ft. square. As this structure is used as a coal loading quay, and stands in 40 ft. of water, it is subjected to heavy loads and severe lateral shocks.

At the naval dock yard at Rosyth, Firth of Forth, Scotland, is found one of the most unique and interesting reinforced concrete docks or piers in existence. It is triangular in shape, 620 ft. long, has been finished recently, and forms the entrance pier to a tidal basin. The outer end consists of seven concrete monoliths with a mass concrete fill in the center, making a solid concrete head, 127 ft. long and 65 ft. wide, covering an area of some 8 300 sq. ft. The rest of the pier consists of twenty concrete monoliths, 25 by 30 ft., supporting a reinforced concrete structure consisting of girders, columns, arches, braces, and deck-beams. The upper decking consists of 3 by 10-in. creosoted wooden joists covered with a creosoted planking arranged so as to prevent any water from collecting thereon. It is a most massive and homogeneous structure, well capable of absorbing any heavy stresses it may receive from warships lying alongside.

In the Bay of Bristol, at Clevedon, where the tide has a rise and fall of some 49 ft., there is another interesting reinforced concrete structure, in the nature of a landing stage. It is 95 ft. long and rests on 22 reinforced concrete piles. The piles, extending up to low-water mark only, were driven through the marl until they reached hard rock. On top of these piles the reinforced concrete landing stage was erected, a structure consisting of columns, beams, bracing, and four different decks or landings.

A reinforced concrete dock of magnitude was recently constructed at Port Talbot. Being designed for coaling operations, it is subjected to heavy loads. The designed load was 850 lb. per sq. ft., the test load being 1 390 lb. per sq. ft., covering an area of 720 sq. ft. The outer row of piling consists of two 14-in. square piles encased in a concrete cylinder, 4 in. thick, and 4 ft. 6 in. in diameter. There are six such columns at the face of the dock. Each of the other two rows consists of eight 14-in. square piles. At this port there are also a number of similar structures, the first having been built in 1907.

It is of interest to note that one of the reasons which influenced the engineers in adopting reinforced concrete for the coaling jetties and wharves at Port Talbot Docks was an extensive fire in one of the docks, due to fuel oil which had escaped from one of the vessels. The oil was ignited through carelessness, and caused a very intense blaze over an area of about 250 by 50 ft. though, fortunately, the oil did not spread out under the wooden structures. With the world-wide use of oil as a fuel for the merchant marine, it is well to consider this danger in American wooden dock structures.

Although the Port Talbot Railroad and Dock Company was among the first to adopt reinforced concrete for wharf construction in England, and so was adversely criticized "for using an alleged untried material in such types of work", the results obtained have fully justified this radical departure from what was at that time the prevailing practice the world over.

Scattered all through other English ports are reinforced concrete docks of various sizes.

At Fleetwood, from 35 to 40 miles north of Liverpool, a reinforced concrete fish and coaling dock of considerable magnitude was completed in 1911. The fish shed section is 1 330 ft. long and 26 ft. wide, that is, the reinforced concrete quay part; the filled-in land behind the quay makes the shed sections some 70 ft. wide in all. The coaling section, of similar construction and carrying a coal-loading traveling crane, is 680 ft. long and 26 ft. wide.

At Harwich, 40 miles northeast of the Lower Thames, is the Parkeston Quay, a reinforced concrete dock structure more than 1 000 ft. long and 51 ft. wide.

On the northeast coast of England, at Newcastle-on-Tyne, a reinforced concrete jetty wharf of extensive size lies along the water-front of a large turret shop.

The port authorities at Dundee and Aberdeen, Scotland, are gradually replacing their worn-out wooden dock structure with reinforced concrete.

On the north coast of Scotland, at Ackergill, stands a life-boat slipway, 194 ft. long, built of reinforced concrete, extending out from the rocky shore into the wide open sea. There are several similar structures in other parts of Great Britain.

A reinforced concrete dock was built in the Shetland Islands in 1910. The head of this dock is 80 by 24 ft., and is reached by a concrete approach, 113 ft. long.

In Cork Harbor and other Irish ports reinforced concrete docks of considerable size have been and are being built.

At Portsmouth, Plymouth, and Cardiff are found the first reinforced concrete docks constructed in England, having been built previous to 1906. They are small, and are used mostly for coaling purposes. Quite extensive reinforced concrete docks, constructed during the last year or two, are found at Newport (Mon.), Swancombe, Gravesend, Portencross, South Bank, Ipswich, Newlyn, and numerous other places.

A number of small reinforced concrete docks or pier-heads, other than those just mentioned, exist in England in connection with shipyards, etc. Such docks are found at Dumbarton Shipyard, on the Clyde, and at several similar establishments.

At first the Hennebique system prevailed in all English reinforced concrete dock construction, but, of late, several other systems have been introduced, the most pronounced of these being the Considère spirally armored concrete piling. Though different types of piles are used in English reinforced concrete dock work, there is a most thorough system of diagonal bracing with each type.

In using the Hennebique or other systems of dock construction, where lateral and diagonal braces of reinforced concrete are put in the structure, it appears that trouble has arisen, and might again arise, from the joints in the bracing system. As the foreign docks built on the Hennebique and similar systems seem to have been a success, it does not perhaps cause as much trouble as it did at first, or would appear to cause. In a concrete structure, of whatever design, built in the water, there is always the danger of cracks below the water line, and these cannot be seen and properly attended to.

In a recent address, Mr. Robert Porter, of *The London Times*, stated that "England is one big port". From the vast number of reinforced concrete docks at present in her harbors, it does not seem amiss to say that some day soon the ports of England will be, figuratively speaking, one big reinforced concrete dock, and it will be impossible to enter that country without passing over a structure of this type. In fact, current English technical publications plainly indicate that there

is hardly a port of any prominence along her coast where reinforced concrete construction is not now being carried on extensively, to the extent of five heavy coal tip docks in one harbor alone, due to the great economy of such structures over their old wooden predecessors, in the way of maintenance expenses.

In a recent rebuilding of the old wooden docks or wharves at Plymouth, constructed 30 years ago, the engineers of the Great Western Railroad have stated that concrete construction was adopted because "the cost would be about two-thirds of the cost of rebuilding in timber." These new concrete docks rest on Considère piles, and were designed for a live load of 400 lb. per sq. ft.

In closing this description of English reinforced concrete dock construction, it is well to take note of a few points in favor of this type, as set forth by an English engineer from his experience in that field:

1st. Easy to build;
2d. Indestructibility;
3d. Small cost of annual maintenance; and
4th. Easy to repair.

Spain.

In Spain a number of interesting reinforced concrete dock structures have been built. At Seville, on the Guadalquivir, 50 miles from the sea, are two reinforced concrete ore-loading wharves of some magnitude. At the outer end of each a series of well-based columns supports an ore-loading car-tipping gear. The ore trains run over an extensive concrete viaduct to this tipping gear, and the ore is dumped from the cars by mechanical means into vessels lying at the head of each dock.

A rather striking reinforced concrete dock structure was built several years ago at Aznal Collar mines, also on the Guadalquivir River. It is 532 ft. long and 17½ ft. wide. Each bent consists of two piles capped with a deep concrete beam. As the bent piles are only 10.8 ft. from center to center, the deck system is cantilevered out about 3¼ ft. beyond the supporting piles on each side. The dock carries an electric traveling crane, the track of which runs along the outer edge of the cantilevered decking. The crane has a capacity of 11 tons at a radius of 39½ ft.

France.

In discussing reinforced concrete dock construction as carried out in France, it is of interest to note that the first reinforced concrete pile was made in France by the noted French engineer, M. Hennebique, in 1896, since which time the reinforced concrete pile industry has grown to vast proportions, being used extensively by almost every civilized nation on the globe. From the successful manufacture and use of concrete piling has grown the construction of reinforced concrete docks. Though reinforced concrete construction seems to have been of a negative nature during the close of the 19th century, except in France, its development, as applied to docks, was due to the influence and perseverance of M. Hennebique, and that too in a country other than France; thus reiterating the old saying that "a prophet is not without honor, save in his own country." The first use of reinforced concrete sheet-piling was made at Sable d'Olonne, Bay of Biscay, France. Though these piles were constructed in 1898, as a reinforcement to an old masonry sea wall, they are said to be in as good condition as when first placed.

To record the most important reinforced concrete docks in France: A concrete pile and retaining wall dock was built in 1902 at Nantes, on the Loire, 40 miles from the Bay of Biscay, and in 1912 was reported to be in good condition. At Arcachon, on the Bay of Biscay, not far from Bordeaux, the French engineers have constructed a reinforced concrete recreation pier, several hundred feet long, with some attempts at the artistic, to harmonize with the surrounding landscape. In constructing a quay type of dock at Dives, near the mouth of the Seine, concrete sheet-piles of the Coignet type were used. They are 7½ ft. from center to center, with a concrete slab closing the gap between them. This type consists of a pile with a sort of two-wing arrangement. A cross-section of the complete pile resembles a very flat T-beam. The piles are of 10 by 12-in. section, 26½ ft. long, and the wings are 5 in. thick, and of such width as to make the whole panel pile 4 ft. wide and 18 ft. deep, the pile part thus projecting about 8½ ft. below the panel part. This dock or quay is 190 ft. long. At Fecamp, 20 miles north of the mouth of the Seine, and exposed to the full sweep of the seas, there is a reinforced concrete dock, built in 1911 on the Hennebique system. The depth of water alongside the dock is 28 ft. at high tide. The dock is well braced by diagonal concrete struts, and

has a concrete deck-slab. Further to the eastward, near the Straits of Dover, at Boulogne, there is another reinforced concrete dock, built in 1906 in connection with a quay dock, the reinforced concrete section being 51 ft. wide and 1 050 ft. long.

As France is not a large maritime nation like England, the development of reinforced concrete docks there has not been so extensive as in some of the other European countries.

Holland.

At Ymuiden a reinforced concrete landing stage was constructed in 1903. It consists of two rows of hollow cylinders of reinforced concrete, 8 ft. 2½ in. in diameter and 28 ft. 8 in. in height, filled with sand. The rows of cylinders are 21 ft. 4 in. apart athwart the deck, but 16 ft. 5 in. longitudinally. The deck consists of the usual concrete beam, slab system, and carries two railroad tracks, being designed to support a 45-ton locomotive. The working face of the dock has a wooden fender-pile protection. With the exception of an extension of this landing stage, no other reinforced concrete dock work standing in sea water has been undertaken in Holland; however, extensive reinforced concrete quay walls, etc., have been constructed during recent years at Rotterdam, a fresh-water harbor on the Rhine.

Italy.

Though rather extensive use has been made of mass and reinforced concrete in the development of Italian harbors, etc., the writer has records of only two reinforced concrete docks built in that country. One is at Pozzuoli, near Naples, a regular Hennebique type of pile construction, with a heavy shear leg derrick at its outer end, and the other at Ravenna, on the Adriatic Sea.

Germany.

The overseas shipping of Germany is confined almost entirely to the two ports, Hamburg and Bremerhaven, the former of which is 85 miles from the sea, so that, compared with England, the number of reinforced concrete pile docks standing in sea water in Germany is exceedingly small. Although the port of Hamburg is equipped with the most modern shipping facilities, most of the docks are of the heavy masonry quay type, and not the long piers so common in America. This

is due to the narrowness of the Elbe as compared with the broad expanse of water-front of other ports. Although the writer's file is lacking in records of reinforced concrete pile or similar docks as having been constructed in Germany, no doubt they exist and are of considerable magnitude. It is hoped that some of the members of the Society may be able to furnish full information concerning them.

As to reinforced concrete dock construction in Sweden, Norway, Denmark, and Russia, the records are extremely meager, perhaps because there are no docks of this type in these countries. Russian engineers, however, have built several concrete docks in the Baltic Sea, there being one at Touapse, a jetty built of reinforced caissons.

Although little, if any, information seems to be available covering the harbor work, etc., of the eastern part of the Mediterranean, in Grecian, Turkish, and Asia Minor ports, it would be plausible to consider that the concrete industry is well known in that part of the world, and that it has been used to considerable extent in eastern Mediterranean ports as well as in the Black Sea, in the construction of not only graving or dry docks, but also reinforced concrete docks and quays.

Africa.

At Alexandria, Egypt, there is a reinforced concrete lighthouse, in 30 ft. of water, exposed to violent storms and heavy seas. Its base is a truncated pyramid 45 ft. high. The slender main tower extends some 33 ft. above the base, its top being about 48 ft. above sea level. At one time this lighthouse was rammed by a 3 800-ton steamer, but it does not seem to have been injured, except that, since that time, it has leaned 4° from the vertical.

At Senegal, on the French West Coast, a reinforced concrete pile dock and jetty was built in 1912. The dock is 242 m. (794 ft.) long, of varying widths, covering an area of 2 500 sq. m. (26 900 sq. ft.). The jetty head is 60 by 10.3 m. (196.8 by 33.8 ft.), with an approach 140 m. (461 ft.) long and 2.65 m. (8.7 ft.) wide, giving the jetty a total area of 1 300 sq. m. (13 990 sq. ft.).

At Cape Town, a long reinforced concrete pier was finished recently. The concrete pile part is 800 ft. long and 45 ft. wide, except for the outer 75 ft., where it is 65 ft. wide. The piles are 14 in. square, 200 being used in the structure. They were driven in groups of four, 13 ft. and 10 ft. from center to center, with a longitudinal clearance of

27 ft. and an across-dock clearance of 18 ft. between each group of four, thus enabling small boats to pass under the dock as well as down the center aisle, as it were. The outer 75 ft. is well braced, the piles being close to one another.

Australia.

Australia and New Zealand are far distant, and thus perhaps few data reach the American engineer as to their developments in concrete. From what is known of several famous concrete structures in that part of the world, these countries are well in the fore as respects reinforced concrete. In fact, at Auckland, New Zealand, there are from four to six reinforced concrete docks, two of which, at least, are of extensive size, *viz.*, a railroad dock, 1 500 ft. long and 240 ft. wide, and Queen's Dock, 1 200 ft. long and 280 ft. wide.

It is of interest, in passing, to take note of a reinforced concrete lighthouse standing in the Straits of Malacca, a two-story closed-in structure, supported by seventeen reinforced concrete piles. On top of the main structure there is an open part consisting of reinforced concrete columns and braces supporting the light and its house.

India.

At Madras three reinforced concrete docks, of the bulkhead type, were constructed in 1911. They consist of a series of concrete piles driven uniformly 8 or 10 ft. apart, depending on the type of construction used. Back of these piles were placed reinforced concrete slabs, in the same way that wooden planks are placed back of and secured to wooden timbers where bulkheads are built to retain earth.

In one type, where the water is only 6 ft. deep and the dock has a freeboard of 10 ft., 18-in. piles, 10 ft. apart, with a penetration of 18 ft., were used. The thickness of the two lower concrete slabs is 12 in., and that of the top slab 15 in.

In another type, where the water is 9 ft. deep and the dock has 6 ft. of freeboard, 15-in. square piles, 8 ft. apart, were used, the concrete slabs being 6 in. thick. The tops of the piles are connected by reinforced jack-arches. In the third type, the piles are 15 in. square, 10 ft. apart, and are tied back to anchors 30 ft. behind the face of the dock. The water is only 4 ft. deep at the face of the dock. Each of these three walls is about 1 500 ft. long, their total length being 4 700 ft.

China.

What is said to have been the boldest undertaking in the construction of a reinforced concrete dock is found at Shanghai on the Whang Poo River, 50 miles from the open sea. The great difficulty encountered in the building of this structure and its accompanying warehouses was the excessive depth of the overlying river silt. The design of the structure itself presents nothing especially new, in comparison with other dock structures, as built in England. The fact that nothing but mud of various degrees of solidity, with layers of impure sand at irregular depths and a thin layer of gravel at a depth of 300 ft., covered the location of the proposed structure for a tested depth of 400 ft., gave a most unsatisfactory soil on which to construct a dock. At a depth of about 400 ft. there was a bed of gravel. The top layer of 25 ft., into which the concrete piles were driven, consisted of a solid crust of stiff sandy clay crossed with old creek beds filled with soft mud, rendering the surface somewhat irregular. The load on the piles had to be carried by the skin friction of this upper layer of supposedly firm ground. From a careful study of the situation, and from deduction obtained from old timber docks built in the same soil, a factor of 500 lb. per superficial foot of pile was fixed as the maximum carrying capacity of the piles, exclusive of their own weight.

The dock structure is 1 160 ft. long and 174 ft. wide, the rear 160 ft. being covered with two steel-frame sheds arranged in such a way as to leave an open space of about 250 ft. between them as an entrance way to the main office building. The decking of the dock was designed for a load of 500 lb. per sq. ft., with a deck load of 350 lb. per sq. ft. for the general structure. In general, the structure consists of a 5-in. deck-slab, deck-beam, and girder system of concrete construction, carried by 15 by 15-in. concrete piers, 15 ft. from center to center, each pier being supported by four 14-in. square concrete piles. At the face of the dock, where there is a large traveling crane, the columns are 18 in. square, and are carried by a group of six 14-in. square piles. The designed load per pier was 45 tons, or 11 tons per pile. The structure has the usual horizontal, cross, longitudinal, and diagonal bracing ties.

Due to the extraordinary conditions under which this dock was built, it is of interest to note a few facts as regards the pile-driving and the final loads carried. The piles were driven with a 3½-ton steam

hammer, the weight of the cylinder, which acted as a dead load on the pile during driving, being 1½ tons. The piston stroke was from 6 to 8 in. The piles had a penetration of about 25 ft., the final set being 1 in. under the last blow. It is stated by the engineer of this structure that "no anxiety was felt as to the future stability of the majority of the piles" under such an excessive set, as it was considered that their supporting power would be increased after the skin of water between the mud and pile had escaped and true skin friction was established. The resistance to driving increased very little after a 15-ft. peneration had been reached, and sometimes was less for a penetration of 25 ft. and above. On one occasion the piles sank so rapidly under the imposed dead weight of the hammer and cylinder that their descent had to be checked by holding up the dead weight of the hammer at the desired elevation. After 4 weeks, this group of four piles was tested with a load of 45 tons, and there was a 1¾-in. setting in 2 weeks, when the settlement ceased. The piles were lengthened 8 ft., and then driven deeper. Though it took 80 blows to re-start the piles, they were finally driven with the same ease as at first. A subsequent test of 45 tons gave a settlement of 1½ in. No additional attempts were made to lengthen any of the piles. When completed, the dock was loaded to more than that for which it was designed (1 000 lb. per sq. ft., by mistake), with no sign of any settlement.

In general, the piles were from 6 to 8 weeks old before they were driven. The range of temperature was about 150°, which appeared to cause hair cracks in winter along the lines where work was stopped in the deck-slab. One of the leading dock engineers of England has remarked that this dock illustrates the great advantages of a concrete decking in its ability to distribute the superimposed loads and thereby assist the piles to carry a heavy burden. The larger part of this structure was moulded on land and put in place on the unit system, as it were, especially those parts below water, except the caps. In connection with this dock, there was also built a quay wall about 495 ft. long and 21 ft. wide, on one side of the property, for the berthing of lighters.

Japan.

The writer has no record of any reinforced concrete pile docks having been built in the development of Japanese harbors, yet he has no doubt that they exist. For 20 years, at least, the Japanese have

been using concrete in breakwaters to an enormous extent, and it is only a part of wisdom to consider that they have used it in other reinforced concrete structures in sea water. Recently, extensive developments have been made in the harbor of Kobe, where four concrete piers have been built, and of a most unique type of construction, *viz.*, reinforced concrete caissons built in the dry on a pile structure on the shores of the harbor, then lifted off this foundation by a specially designed depositing dock, carried into deep water, and put afloat by sinking the depositing dock and withdrawing it from beneath the concrete caissons. The caissons were then towed to and sunk in their proper places in the quay wall. Each caisson, 119 ft. long, from 35½ to 41½ ft. high, and from 24 to 30 ft. wide, has twenty open-top cells, and weighs from 1 900 to 2 400 tons.

Philippines.

It may not be exact to regard the Philippines as a foreign country, so far as concerns engineering work now being done, yet, in this paper, it has been classed as such, and note is here made of the reinforced concrete docks in those waters. The first dock of this type was built in Cavite at Manila in 1902, on what is known as the "Cushing System of Piers", *viz.*, cast-iron cylinders surrounding wooden piling and filled with concrete, the deck system consisting of steel girders and beams covered with a concrete slab. The dock is 408 ft. long, 75 ft. wide, and contains thirty-six cylinder piers 6 ft. in diameter. The structure was built by the United States Navy Department in connection with a coaling station.

Two docks of a similar design were built in 1906 by the Navy Department at Manila. One of these is 650 ft. long and 110 ft. wide; the other is 600 ft. long and 70 ft. wide. There are nearly 300 cylindrical piers in the two structures.

The Navy Department has also constructed two reinforced concrete docks at the Naval Station, Olongapo. One of these is 332 ft. long and 45 ft. wide; the other is 300 ft. long and 60 ft. wide. The piers or columns of both consist of hollow concrete cylinders, 2½ ft. in diameter, filled with concrete, and 12 ft. apart longitudinally and 18 ft. transversely. The decking consists of I-beams, encased in concrete, and these support a suitable concrete slab. There are light diagonal ties at each bent, running from the base of each outer column to the

top of each inshore column, but there does not seem to be any other lateral bracing.

Another reinforced concrete dock has been finished recently in the Philippines, at Iloilo, about 330 miles south of Manila. It runs along the face of an old sea wall, and consists of a series of transverse girders, 10 ft. from center to center, with a 12-in. concrete deck-slab. The outer end of each girder rests on a hollow cylindrical column, and the land ends are seated on a concrete pedestal built up from the natural slope of the bank. At the face of the wall, arches are inserted between the girders in order to provide additional strength to resist lateral stresses, as well as to give to the dock an artistic finish.

South America.

In view of the extensive harbor developments and the magnificent systems of docks, etc., in some of the ports of the leading South American countries, it is to be regretted that more general information concerning them is not readily available for the engineers of the United States. These engineering undertakings and port developments stand out as the most prominent of such world undertakings, especially at Buenos Aires, which city is conceded to have the finest shipping and port facilities in the world, with Montevideo a close second. It has been stated that almost all South American docks of any magnitude are being constructed of concrete, though the percentage of reinforced concrete structures among them is unknown to the writer.

In the foregoing the writer has attempted to outline the most important concrete docks, etc., constructed up to date in European and other countries, without any attempt to go into the technical side of their design, though some of them are more in the nature of semi-concrete and concrete steel docks than full concrete structures. The list is far from complete, but gives a general idea of the extensive use of reinforced concrete in dock construction in foreign ports.

Part II.—Reinforced Concrete Docks in the United States.

In recording what has been accomplished in the construction of reinforced concrete docks in North America, including Porto Rico, Cuba, the Canal Zone, and Canada, such undertakings are so few and of such recent date, compared with those in European countries,

that the art of building reinforced concrete docks there may be said to be hardly beyond its infancy, especially as regards out-and-out docks or piers, as the American usually understands the word, *viz.*, long structures running out from the shore in such a way that vessels can lie on each side. Unfortunately, it will be necessary to make note of some failures among North American concrete docks.

Atlantic Coast.

Boston.—The first concrete dock built in Boston Harbor has perhaps caused more discussion as to the feasibility of using concrete in sea water than any other American structure of this type, and, therefore, is far-famed in itself. This dock, or pier, was built at the Charleston Navy Yard about 14 years ago. The first section, consisting of a long, straight wall, was built in 1899-1900, without any resort to a coffer-dam. The other two sections, consisting of plain reinforced arches, 20 ft. wide, with spandrel walls, were constructed in 1901, the space between the two walls being filled with earth and stone. The first section was built of 1:2:3 concrete throughout, and it was all placed in the wet, with an open-top bucket. The second and third sections were of different mixtures, the main body being of 1:3:6 concrete, the outer 2 ft. consisting of a 1:2:4 mixture, and the whole exposed surface was faced with 3 in. of 1:1 mortar. In these two sections the concrete was placed very dry, as was the practise at that time.

During 1913 one of the Boston public docks, Pier No. 5, was partly rebuilt in concrete. This dock consisted of a timber platform deck, 50 ft. wide and 1 150 ft. long, on each side of a solid earth fill. The new wooden piles needed were driven, and all the piles, new and old, were cut off at mean high water and capped with a deep reinforced concrete beam running athwart the piles, in the way of curtain-walls. These curtain-walls support two longitudinal reinforced concrete beams, the third or inner beam resting on the earth retaining wall. There are also two additional concrete beams under the track along the outer face of the dock, with special supporting piles. On top of the curtain-wall and longitudinal girders is laid a reinforced concrete floor-slab with a 2-in. bitulithic top. Wooden longitudinal tie members are run from top of pile to top of pile under the concrete curtain-walls, the whole timber structure being well braced with piles and wooden ties. To provide against accident from disintegration

of the lower part of the curtain-walls, due to frost action or other cause, cast-iron columns, 30 in. long, are attached to the top of the piles and made integral with the curtain-walls, thus guarding against weakness in the concrete, rather than in the piles, as done in other harbors.

New York Harbor and New Jersey Coast.—A reinforced concrete dock of an experimental nature was constructed at Ellis Island, New York Harbor, in 1911. It is a rather small structure, 30 ft. wide and 50 ft. long, resting on thirty-six driven concrete piles 18 in. square. The piles were made of different mixtures, for experimental purposes, and various kinds of water-proofing were used in order to determine their efficiency under the same conditions. This was the first complete concrete pile and deck dock built in New York Harbor.

During the past seven years a semi-concrete type of dock has been under development in New York Harbor, *viz.*, wooden piling, wooden caps, and concrete decking. In one dock, on the New Jersey side of the Hudson, the caps on top of the piles are also of concrete. In the final type, as worked out by the Department of Docks and Ferries,* the concrete slab rests directly on wooden caps secured to the tops of the wooden piles, a genuine flat slab between bents, the entire timber floor system being wholly eliminated. At present some 25 or 30 semi-concrete docks have been built on this system in New York Harbor. It has been stated authoritatively that they have proved a great success.

In building two semi-concrete docks at the Brooklyn Navy Yard a few years ago, the objectionable features of docks of the foregoing type—*viz.*, part of the wooden pilings and bracing exposed to wet and dry conditions between low water and the decking, and the wooden cap as an additional temporary item helping to support a permanent structure—were eliminated. In the two Navy Yard docks the wooden piles were cut off a little above low water and capped with a wooden grillage. Pre-moulded concrete columns, mixed with water-proofing compounds, were set on and dovetailed into the caps, and a concrete girder-beam and deck-slab system was worked over the tops of the columns. The wearing surface consists of a creosoted wooden block pavement. Down each side of the dock there is a standard-gauge railroad.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 503.

A small concrete dock was constructed at Glen Cove as a yacht-landing, in the winter of 1909-10. It consists of eight reinforced concrete rock-filled caissons, supporting an overhead footbridge, the total length of the pier being about 330 ft.

Long Branch.—At Long Branch a Hennebique type of concrete pile dock was constructed in 1911, running some 848 ft. out into the Atlantic Ocean, as a boat landing and recreation pier. At present the pier is only 75 ft. wide, except for an 80-ft. length at its outer end, where its width is 150 ft., the intention being to make the whole pier of that width at some future time. The deck is 22 ft. above low water. The piles are 16 ft. from center to center longitudinally, but 20 ft. from center to center across the pier, except the outer two rows, which are 15 ft. from center to center. Most of the piles are of hollow cross-section, 22 in. external diameter, 13 in. internal diameter; the penetration was about 22 ft. To provide sufficient impermeability, the shells of the piles were made of 1:1½:3 concrete, the fill being of a weaker mixture. Apparently, no cross-bracing system was used, the outer end of the pier being stiffened laterally by inclined bracing piles at regular intervals.

Atlantic City.—At Atlantic City the famous steel pier was widened and protected in 1906 by the use of concrete. The original pier was founded on steel-pipe piles resting on cast-iron disks, a type of construction quite common during the last part of the 19th century. The pier extends out into the Atlantic Ocean a total length of 1 600 ft. In rebuilding the pier all the original metal work was encased in concrete and the pier was widened on both sides, 12-in. and 25-in. reinforced concrete piles, with enlarged footings, being used. The smaller piles were pre-moulded vertically, on a small platform, each pile at its final location. After hardening sufficiently they were lifted off their platforms and jetted into place through from 8 to 14 ft. of sand. The larger piles were given a penetration of 16 ft. The lower 12 ft. of these piles were pre-moulded on a platform, a water-tight iron casing was secured to the upper end, and the whole was jetted into place. The dry caisson was then filled with concrete up to the proper level, the maximum total length of the 25-in. piles being 52 ft. In protecting the original piles, concrete shells were cast around them, with sufficient interior clearance, and the space was afterward filled with grout.

After the concrete work of this structure had been in the sea water for 6 months, the piles became coated with a sort of gelatinous matter which seemed to act as a most excellent protective coating against any deterioration. The same peculiar action has also been noticed in California.

Although not exactly a dock, it is of interest to note the concrete pile Boardwalk at Atlantic City. Not only is it necessary to guard against dry and wet conditions at such resorts, but the fine sands act like a sand-blast when driven like snow before the wind. In 1908 part of the old wooden structure was rebuilt with 16-in. concrete piles, supporting a concrete cap, and that in turn carried the wooden decking.

Baltimore.—It is perhaps at Baltimore that the most extensive reinforced concrete docks on the Atlantic seaboard have been built. Although the water in Baltimore Harbor may not have the same density of salt as in ports nearer the sea, these docks, thus far, have shown no sign of deterioration, though at times subject to frost action. Three of these piers are of a back-filled concrete bulkhead type, and are not docks resting on piles. Pier No. 4 is 978 ft. long and 220 ft. wide; Pier No. 5 is 1 245 ft. long and 200 ft. wide at the shore end, but 243 ft. wide at the water end; Pier No. 6 is 1 456 ft. long and 93 ft. wide at the shore end, but 212 ft. wide at the water end; all were built in 1908.

In general, these three docks consist of a series of oval-shaped concrete cylinders 25 ft. apart along the face of the docks, and sunk to about 25 ft. below low water. Along the face of the cylinders, and just above high water, there is a concrete-encased iron girder, tied back to a deadman some 28 ft. in the rear of each cylinder. A row of concrete sheet-piling was driven back of the girders to form a vertical retaining wall, the upper ends of the sheet-piling bearing against the girder, and the lower ends being driven into the muddy bottom. A horizontal box-girder encased in concrete runs along the upper face of the dock, supporting the outer edge of the concrete curb slab, on which are laid the paving blocks. The cylinders are tied together in certain cases by ties extending entirely across the docks. The face of each dock is protected by wooden fender-piles, 8 ft. apart. Another concrete dock has been completed recently in Baltimore by the Harbor Commission, the details of which are lacking. In the

same harbor is found a concrete bulkhead dock, built for a private corporation—a reinforced concrete sheet-piling structure capped with a concrete girder tied back to deadmen by reinforced concrete ties.

At Sparrows Point, near Baltimore, a reinforced concrete ore dock, 600 ft. long, was built in 1911. It consists of two parallel concrete walls, about 46 ft. apart, *viz.*, (1) a sheet-pile bulkhead on the water-front capped by heavy concrete girders with a cantilevered shelf, as it were, on the outer face, running the full length of the bulkhead; (2) a heavy retaining wall in the rear, the two walls being tied together by reinforced concrete ties about 30 ft. apart. The back wall, resting on wooden piles, not only acts as a deadman for the outer wall, but affords a means for carrying one track of the large, heavy, ore unloading crane that straddles the filled-in space between the two walls, the front track of the crane running along the outer wall. The dock face is protected by a substantial system of fender-piles and wales with heavy helical car-springs at each buttress of the face wall.

Norfolk.—In constructing the Virginian Railway Coaling Terminal, at Sewells Point, in 1907, it was not practical to carry the massive steel superstructure on creosoted piles. In place thereof groups of wooden piles were driven and cut off 1 ft. below the mud line. On top of these piles were built monolithic concrete piers, of pyramidal shape, to 4 ft. above high water. All the concrete work was done in the dry, inside a coffer-dam. These piers are reported to be in as good condition as when first built.

Brunswick and Charleston.—Perhaps the most extensive development of concrete dock construction, combining concrete piles with wooden decking, is found at Brunswick, Ga., and at the U. S. Navy Yard, Charleston, S. C., built in 1906.

The Brunswick terminal consists of two piers, 500 and 900 ft. in length, respectively, and each is 140 ft. wide; there is also a coaling pier about 300 ft. long. The 16-in. bearing piles, of pre-moulded concrete, are 12 ft. from center to center each way. They are from 30 to 51 ft. in length, with the lower 10 ft. tapering to 8 in. The piles have a penetration of 40 ft. Their upper ends are corbeled out to support the double 8 by 16-in. wooden caps. The decking consists of 6 by 14-in. stringers and a 3-in. flooring. Each bent is well braced with creosoted wooden cross-bracing.

The Charleston Navy Yard dock is 60 ft. wide and 520 ft. long, and of the same type of construction as the Brunswick structures. The piles, 10 ft. from center to center each way, are 18 in. square, instead of 16 in. square, and have an 8-ft. taper to 12 in. square at their lower ends, thus giving a heavier structure than those at Brunswick. The test load on the Charleston dock was 30 tons per pile for 48 hours, though the specification required only 20 tons, or 400 lb. per sq. ft.

The outer row of piles in these docks consisted of three creosoted yellow pine sticks, two of which were driven on a batter; all were bolted together to afford sufficient protection to the dock in the form of a fender-pile system.

During the building of the Brunswick dock it was rammed by a large steamer. Although a number of the pine piles were broken, it has been stated that the concrete piles withstood the shock successfully.

Savannah.—A rather unique type of concrete dock was built at Savannah, 17 miles from the sea, in 1913. The design seems to contain many of the excellent features of the Ambursen dam. This dock consists of a series of pile bents athwart the dock supporting reinforced concrete brackets of triangular shape, the brackets in turn supporting a concrete deck-slab sloping down and toward the rear of the structure. This deck was afterward back-filled and finished off with a suitable working face. The dock forms a water-front structure, and is protected by wooden fender-piles.

Jacksonville.—In the construction of a semi-concrete dock at Jacksonville, the Braxten concrete pile was used, with very satisfactory results. In another case the Ripley concrete-encased wooden pile was adopted.

Key West.—A reinforced concrete quay wall dock, 1 589 ft. long, was completed at the U. S. Navy Yard at Key West in 1912. The main wall consists of a series of pre-moulded concrete pile bents capped by a concrete girder and a deck-slab 40 ft. wide on top. From the inner edge of the deck-slab a sloping concrete apron runs down to the top of a row of sheet-piles which forms a retaining wall for the reclaimed land. The piles are from 16½ to 20 in. square, and vary from 25 to 60 ft. in length. The bents are 10 ft. apart, with the same spacing for the piles, each bent having six piles. The face of the dock is protected by a system of creosoted fender-piles placed midway between each bent.

Port Arthur.—A reinforced concrete pile-bent dock, 1 050 ft. long and 25 ft. wide, was constructed at Port Arthur, Tex., in 1911-12. In general, the piles are 16 in. square, 44 ft. long, and 5½ ft. from center to center. The pile bents, of five piles each, are about 23 ft. apart, and are capped with a reinforced concrete girder. Five concrete beams, running from bent to bent, and a 4½-in. concrete slab, form the deck structure. The dock is tied back to the concrete trestle built for carrying the railroad tracks in the rear of the dock. No provision is made for any spring or other device to take up the impact forces on the fender system, as it is believed that the wooden fender-piles will afford sufficient elasticity to prevent any injury to the dock from this source.

Cuba.—Two reinforced concrete docks, 620 and 670 ft. in length, respectively, and 160 ft. wide, were built in 1911-12 at Havana, the depth of the water varying from 12 to 40 ft. Each consists of a concrete floor-slab resting directly on concrete caps placed on top of clusters of from four to eighteen reinforced concrete piles, the clusters being about 23 ft. from center to center in each direction. The concrete piles, 18 and 20 in. square, were designed for a load of 32 tons each. The design of the floor slab would indicate a cantilever effect longitudinally between each row of longitudinal piling.

One of the railroad companies of Cuba, also, has built a reinforced concrete dock at Havana, for coaling purposes. The structure, which is subjected to very heavy loading, rests on Chenoweth concrete piling, and was but recently finished.

Haiti.—In constructing a reinforced concrete dock at Port au Prince, during 1913, the Ripley type of concrete wrapped wooden pile was adopted. This dock has a total length of 2 326 ft., varying in width from 24 to 60 ft. The piles are 10 ft. from center to center, longitudinally and transversely, and are capped by heavy concrete girders of rectangular section for the inshore end of the dock, otherwise by arched girders. The deck system consists of a series of reinforced concrete beams supporting the concrete deck-slab, built with a crown, in order to shed water. The dock is protected by a creosoted fender-pile system.

Panama and Canal Zone.—The United Fruit Company in 1909 built a combined reinforced concrete and wooden pile dock at Bocas del Toro, for the docking of fairly large steamers. The wooden piling is surrounded by a 4-in. concrete shell up to about 1 ft. above the

high-water line. The piles are extended up to the deck as reinforced columns, with a concrete beam and deck-slab system. Up to the present time the dock is said to have given good results.

In the Canal Zone the U. S. Army Engineers have constructed a reinforced concrete dock, 706 ft. long and 55 ft. wide, for unloading timber. There are fifty-five concrete piers or columns, 8 ft. in diameter and about 80 ft. long arranged in two rows, 35 ft. from center to center across the dock, and 30 ft. from center to center longitudinally, and built in the form of hollow reinforced concrete sectional cylinders. After these cylinders had been sunk to bed-rock, they were filled with concrete, being reinforced vertically with eight rails. On top of the columns there is a concrete girder, deck-beam, slab system. The girders are about 5½ ft. deep, the beams about 4½ ft. deep, and the slab 6 in. thick. The railroad track runs over one row of columns. The floor system is designed for a load of 400 lb. per sq. ft., with a concentrated load of 105 tons over the track beams. The depth of water for a mean sea-level tide is 40 ft., the total fluctuation in the tide being 20 ft.

Two other concrete docks of extensive size are now in course of construction in the Canal Zone, with still more to follow.

Pacific Coast.

On the long stretch of our Pacific Coast, perhaps is found the greatest development of reinforced concrete dock construction in the United States. This section is making vast harbor improvements in anticipation of the opening of the Panama Canal.

San Diego.—At this most southern port on the California Coast an extensive reinforced concrete dock is now under construction. It consists of two parts, *viz.*, the dock itself, 800 ft. long and 130 ft. wide, and a quay wall or bulkhead, 2 675 ft. long and 25 ft. wide. Wooden piles are driven into the soil and cut off "at any point between mean low water and 18 ft. below city datum." Each of the 42-in. concrete columns encases one wooden pile and supports a system of structural deck-beams, a concrete slab covering the whole. The columns are 15 and 13 ft. 4 in. from center to center. The entire structure is protected by a wooden fender-pile system having the so-called San Francisco type of steel spring shock-absorbers.

San Pedro.—In connection with extensive port developments at San Pedro, a semi-reinforced concrete dock was recently completed in the outer harbor. It consists of pre-moulded concrete piles, 10 ft. from center to center in each bent, the bents being 16 ft. apart. The tops of these piles are corbeled out to support two 10 by 16-in. wooden caps, which in turn support the wooden floor joist and wooden decking. The piles are tied together with a wooden cross-bracing system above mean high tide. The structure is also stiffened against lateral blows on its face by inclined bracing piles. The wooden pile fender system has a car-spring to assist in taking up lateral forces. The dock is of the quay type, 48 ft. 6 in. in width, the total pier head frontage being 12 000 lin. ft. A railroad runs parallel to the inner edge on the inshore fill.

Redondo.—It is of interest to take note of the ocean pier at Redondo, Cal., though it is not a dock. It extends some 637 ft. out into the Pacific Ocean, and supports the intake pipe (for cooling purposes) of a power station. The pier consists of concrete pile bents, 20 ft. apart, each bent having four piles. As considerable surf runs at times under this pier, the outer bents have an extra outside pile driven with a batter of 2 in. per $\frac{?}{?}$ ft. The piles consist of a thin steel shell, 18 in. in diameter, closed at the lower end. After the steel cylinder had been driven to the proper depth of penetration, the reinforcement was inserted and the cylinder was filled with concrete. The piles of each bent have a structural steel cap encased in concrete, with a system of diagonal bracing in a horizontal plane connecting the tops; there is also a longitudinal system of ties at the tops of the piles, above the reach of the water. This structure, as a whole, is said to possess considerable elasticity.

Long Beach, Cal.—In 1907 a concrete pile pier was built at Long Beach, extending some 1 300 ft. out into the ocean. The head of the pier is 100 ft. long and 300 ft. wide, the approach being 1 299 ft. long and 32 ft. wide. The deck is 30 ft. above mean low water, so as to be kept clear of the 24-ft. waves which at times roll in from the ocean. The piles and columns are 4½ ft. in diameter, and are arranged in bents. Under the head of the pier the bents and piles are 16 ft. from center to center. The approach bents are 20 ft. apart, with two columns each. The columns are sunk from 10 to 18 ft. into the sand, in order to be absolutely safe, as it is said that at times the undertow

digs out the sand to a depth of 13 ft. The pile caps consist of steel I-beams, on top of which rest wooden stringers carrying a wooden decking.

Santa Monica.—At Santa Monica, an ocean pier, built in 1908-09, extends 1 600 ft. out into the ocean. It was built on driven concrete piling, ranging from 14 to 22 in. in diameter, in lengths up to 75 ft., with from 16 to 20 ft. penetration. Although the pier was built primarily to support the outfall sewer carrying the sewage effluent to a point far seaward, it is also used for recreation purposes. The pier is about 35 ft. wide at the deck line, with three platform spaces of 43 by 89 ft. at intermediate points and at the end. The bents are 20 ft. from center to center and consist of three piles, the piles being 13 ft. 6 in. from center to center. Each bent has a concrete cap on which rest the wooden joists covered with 2-in. planking, and the latter is covered with a 3-in. wire-mesh concrete slab, having the proper pitch to carry off the water. The bents are tied together by three longitudinal reinforced concrete tie-beams running from top of pile to top of pile. The piles are bulb-pointed.

San Francisco Bay.—Although the dock engineers of New York City have developed a type of semi-reinforced concrete dock, *viz.*, a wooden pile structure supporting a concrete slab, especially adaptable to local conditions, the dock engineers of San Francisco have developed a type of full reinforced concrete dock based on wooden sub-piling, concrete column piers, steel or concrete deck-beams, and concrete floor slab, the concrete encasing the steel beams and the floor being made monolithic, with details varying to suit special conditions. Although the type as worked out presents no difficulty in the way of construction, outside of building the main columns, that part of the work has been done successfully, but with considerable difficulty. The mud line at the bottom of the bay is said to be approximately level, yet, at the outer ends of some of the piers there is a depth of only about 18 in. of mud over the rock; at the shore end, however, there is a depth of 35 ft. of mud. Piles can be driven to a rock bearing in some places, but it is impossible to use wooden piles throughout. Along a portion of the water-front, where it is not possible to reach the rock, there is a hard soil capable of bearing from 4 to 6 tons per sq. ft., thus doing away with the necessity of any sub-piling.

The method used in building the column piers is to sink a hollow

steel caisson, of such length that it will not be overtopped by the water, dredge out the interior to the desired depth, and build the reinforced column in the dry. In some cases the columns rest on solid rock, in others, wooden piles have to be driven inside the cylinders to obtain the necessary bearing support. The size of the columns varies according to conditions. In two docks built in 1910, 140 ft. wide and 780 ft. long, where the mud covering the rock was less than 50 ft., the columns were seated directly on the rock. Where the mud is more than 50 ft. deep the columns rest on five 15-in. wooden piles driven to refusal, the piles being cut off 35 ft. below the water line and encased by the concrete columns to that height. The columns are 6 ft. in diameter to a height of 7 ft., and then 3½ ft. to the top.

In laying out a vast dock improvement proposition at Fort Mason, San Francisco, the Government has planned for the immediate construction of three docks of the usual San Francisco type, each to be 500 ft. long, two 81 ft. wide, the third 118 ft. wide. The concrete columns are to be supported by groups of seven wooden piles driven in a circle 6½ ft. in diameter, and 18½ ft. from center to center each way. The piers are to be 8 ft. in diameter up to 12 ft. above the dredge line, and are then to be reduced to a diameter of 4 ft. for the remainder of their length. The wooden piles will extend some 11 ft. up into the concrete columns, the bottom of the concrete being well below the mud line. In building the first of these docks, an attempt was made to construct the column forms of 4-in. staves, sufficiently reinforced with bands, and sink them into position by driving. The method did not prove a success, and resort was made to the steel cylinder caisson method, as described previously.

Up to 1911 there were only four modern reinforced concrete column docks under the control of the San Francisco port authorities. Since that time they have added largely to the number by replacing some of the older wooden pile docks with reinforced concrete structures. The first addition was Pier No. 17, 800 ft. long and 126 ft. wide, with suitable railroad track accommodations. It consists of wooden piles protected by concrete shells, the deck-beams being of structural steel encased in concrete, and the stringers and decking are of timber—a sort of semi-concrete pile semi-concrete dock.

The next docks reconstructed were Piers Nos. 26, 28, 30, and 32, all of the same type, having reinforced concrete columns resting on

the hard bottom, without any piling, with a complete system of reinforced concrete deck-beams, girders, and slabs. These docks are equipped with up-to-date cargo-handling machinery.

The next addition was Pier No. 39, 150 ft. wide, this being in process of construction at the present time. The concrete columns rest on groups of from 4 to 10 wooden piles, the entire deck system being of reinforced concrete.

In another type of construction at San Francisco the wooden piles are wrapped with wire fabric, or otherwise, and a concrete shell is placed around them after the piles have been driven to place. This method, apparently, has proved successful, though it must be carried on in such a way that the concrete can be poured, set, and hardened in the dry, and not in sea water, if permanent results are to be obtained.

Recently, the City of Oakland, Cal., built a genuine reinforced concrete pile dock, 295 ft. long and 124 ft. wide, standing in 30 ft. of water. The piles are of pre-moulded concrete, 16 in. in diameter, octagonal, and of a 1:1½:3 mixture for a distance of 5 ft. from the top, the remainder of the pile being of a 1:2:4 mixture. The bents and piles are approximately 10 ft. apart. Each row of piles has a concrete cap or girder running athwart the dock, the deck-beams and deck-slab being also of concrete. For lateral stiffness, 12-in. concrete curtain-walls were built at three points in the dock, for about one-third of its width, between the piles in three bents.

Portland.—Being 112 miles from the ocean, on a fresh-water river, it is possible to use wooden piling at Portland, in the construction of docks. A massive concrete dock terminal, now being built by the city, consists of four concrete warehouses along the water-front. The dock part of this project as designed consists of a reinforced concrete platform, 1 030 ft. long and 100 ft. wide, 32 ft. above low water, resting on wooden piles driven in groups and cut off at about mean low water. Resting on each group of wooden piles, 20 ft. from center to center, are the reinforced columns supporting the upper platform, composed of steel I-beams encased in concrete, and a concrete floor-slab. For a length of 300 ft., a low-level deck, 14 ft. below the main deck, is provided. As the Columbia River is subjected to high- and low-water stages, due to floods, it was necessary to provide this lower platform for use by river steamers during low-water periods. The rise

and fall of the Columbia River attains a maximum of about 28 ft., though 18 ft. is about the average, all based on mean low-water level. Thus the lower platform will seldom be under water.

Puget Sound.—Though there are several large shipping ports on Puget Sound, up to the present time, no reinforced concrete dock construction of a commercial nature has been undertaken in these waters. As lumber is so plentiful in that part of the country, it is only natural that such a section should be one of the last shipping centers to take up the building of reinforced concrete docks; but as the destructive teredo is very active there, the engineers of the Northwest are beginning to seek a more stable type of construction than creosoted wooden pile docks, especially the United States and Canadian Governments, and some of the railroads, because "in Government [and publicly owned] docks, a small saving in first cost is of minor importance, but weakness and frequent need of repairs are well nigh intolerable. On the other hand, in private ownership of docks a saving in first cost is usually of serious importance, while the cost of maintenance, repairs, etc., is met by earnings of the dock and is less felt", unless they become so excessive as to make a concrete proposition more economical in the long run.

At the U. S. Navy Yard in Bremerton, Wash., a reinforced concrete dock, consisting of concrete columns, steel and concrete beams, and a concrete slab, was completed in 1912. It is 402 ft. long and 60 ft. wide. The columns are 3 ft. in diameter, with a flared-out footing to a diameter of 6 ft., 16 ft. from center to center, each way, there being four columns to each bent. The caps and girders over the tops of the columns are of I-beams encased in concrete. A concrete beam is run midway longitudinally between each of the four steel beams. The I-beam caps are cantilevered out 6 ft. on each side of the dock. The columns were built as hollow concrete cylinders with a 3-in. shell of 1:2 mortar, and filled with a 1:2:4 concrete after being sunk to place. As hollow cylinders, they avoided any coffer-dam work. The dock has a standard railroad track down each side over the outside columns, also an ordinary wooden fender system.

A still heavier concrete dock is now in course of construction at the same Navy Yard. It is 490 ft. long, 80 ft. wide, and is designed for a load of 600 lb. per sq. ft. The approach of the dock consists of a wooden pile structure, 210 ft. long, of triangular shape. As designed, the structure is supported by sixty-eight concrete columns, 4 ft. in diameter,

with an 11-ft. base (of the same type as in the dock just mentioned), 20 ft. from center to center athwart the dock, and 30 ft. longitudinally. The columns are capped by extremely heavy reinforced concrete beams which support a series of built-up structural beams, about 36 in. deep, carrying a thick concrete floor-slab. The side of the dock is cantilevered out 8 ft. beyond the columns. A standard railroad track runs over the center line of the outside rows of columns on each side of the dock. On account of the nature of the soil, some of the piers rest on sub-piling, others, nearer the shore, on hardpan.*

Vancouver.—The Great Northern Railroad has very recently completed an extensive reinforced concrete dock of the quay type, in connection with a new terminal it is building at Vancouver. The total width of the terminal dock is 302 ft., the concrete dock proper being 456 ft. long, and 50 ft. wide on each side of the terminal, the space between being a rock and earth fill, with a proper rip-rapped slope at its faces. Each concrete dock structure consists of nineteen reinforced concrete columns, 4½ ft. in diameter, 25 ft. from center to center, parallel to the face of the dock, resting on the rock stratum that underlies the location of the terminal. These columns support a heavy longitudinal concrete girder into which are tied heavy cross-girders, 12½ ft. apart. The cross-girders are cantilevered out beyond the longitudinal girder about 16½ ft., and their inboard ends rest on two driven concrete piles 34 ft. back from the center of the columns. Four concrete beams of suitable size are run longitudinally with the dock. The entire girder and beam structure supports a concrete slab. The railroad track is over the longitudinal girder running from column to column. The two parts of the dock are tied together by suitable concrete beams running across the interior fill. The terminal is considered to be one of the most substantial and up-to-date structures on the Pacific Coast. The dock is well protected by a wooden fender-pile steel-spring system.

Great Lakes.

Although extensive use of concrete has been made in some of the ports of Lakes Superior, Michigan, Huron, Erie, and Ontario, in the construction of massive ore-docks, it is not proposed to discuss that particular phase of the subject in this paper. These docks are of

*In the actual construction the built-up structural beams were replaced by reinforced concrete.

excellent design, some built on concrete piling, others on wooden piling or cribs. Concrete piles in fresh water are not subjected to the same deterioration as those in salt water. On the other hand, they have to withstand extremes of temperature, frost and ice, in addition to severe treatment, due to the heavy traveling machinery above them.

Some of these docks will be discussed, in order to bring out the important features of their special design.

Chicago.—One of the first attempts at using concrete for dock work on the Great Lakes was made at South Chicago, in the winter of 1898-99, in the rebuilding of an old wooden quay dock wall. The concrete structure is 1 680 ft. long, and consists of a heavy mass concrete stepped-back wall, 10½ ft. high, 8 ft. wide on top, and 18 ft. on the bottom, supported by a timber structure consisting of three rows of piling cut off 3½ ft. below mean water level, with longitudinal caps crossed by a heavy wooden grillage. A timber sheet-piling bulkhead under the face of the wall acts as a retainer for the slag fill. On one occasion the wall was rammed by a large steamer, but suffered absolutely no damage. The damage to the steamer, however, was rather extensive. A year or so later, a similar quay dock wall, 2 300 ft. long, was constructed at the same steel plant.

Due to the decay of a long wooden water-front bulkhead on the Chicago River, it became necessary to replace it. This was done by constructing a reinforced sheet-pile bulkhead almost ½ mile in length, capped by an I-shaped concrete beam, some 3 ft. wide and 5 ft. high. This beam rests on pre-moulded piles, which are 20 ft. apart and are secured to buttresses, also 20 ft. apart, which run back about 12 ft. from the sheet-piling, the land ends of the buttresses being supported by other piles. The buttresses are also tied back to deadmen some 35 ft. back from the wall. As the bulkhead has about 18 ft. of water along its front, vessels can dock alongside.

Marquette.—The most extensive reinforced concrete ore-loading docks on the Great Lakes are without doubt at Marquette, Mich., and were built in 1912. The substructure consists of a heavy concrete slab and facing-walls, similar to a channel beam placed on its back, 1 500 ft. long and 60 ft. wide, resting on wooden piling. Under the face of the walls there is a wooden sheet-piling bulkhead which retains the sand fill around the piles under the dock structure. The depth of

the concrete web is 3 ft., the walls that correspond to the flanges being 9 ft. high. The substructure supports a very massive reinforced concrete superstructure for loading ore into vessels, the whole dock being a very substantial and shock-resisting structure. A steel plate is worked along the face of the two concrete walls from 6 in. below the water level to 3 ft. above it, to prevent disintegration of the concrete due to frost and ice action at the water level.

Two Harbors.—At Two Harbors there is an ore-loading dock consisting of a steel superstructure supported by a mass concrete wall running along each face of the dock, and tied together with concrete beams at regular intervals. The concrete walls rest on wooden piles with a wooden sheet-piling bulkhead to retain the interior fill. The substructure is 1 400 ft. long and 52 ft. wide.

Detroit.—The concrete ore-loading dock at Detroit is 200 ft. long. It consists of three bearing walls of a T-rail shape, 9 ft. deep, running parallel to each other, the two outer walls being 28 ft. apart, the third or back wall being 173 ft. from the middle wall. The three walls rest on a double row of oak piling cut off 3 in. above the water level. Being capped by the concrete walls, no part of the wooden piling is exposed to the air. The outer wall stands in about 10 ft. of water, the middle wall in 3 ft., and the rear wall is far back on the dry land. The two outer walls are tied together by reinforced concrete beams, 5 ft. deep, at intervals of 10 ft., with cantilevered brackets on the water face of the outer wall opposite the tie-beams. The concrete deck-slab on top of the brackets is 12 in. thick, but only 6 in. thick between the two outer walls. The dock has a suitable wooden fender-pile system.

Between the middle and back walls there is a 12-in. reinforced concrete floor-slab, supported on a series of piles, 5 ft. from center to center each way, stated to have been designed for a load of 6 800 lb. per sq. ft., or 85 tons per pile. The ore floor is tied into the middle and back bearing walls opposite each cross-beam in the dock proper. The three walls support an ore bridge tower, used for unloading ore from vessels to cars on the track just behind the middle wall, or into the ore bin between the middle and back walls.

Toledo.—At Toledo, Ohio, there is an ore-unloading dock, consisting of a plain concrete wall running along the water-front, resting on wooden piles, and tied back to deadmen some 100 ft. in the rear. The

land is reclaimed back of the middle wall, a row of sheet-piling under the river wall retaining the earth in place. Parallel to the river there are three other walls to support the legs of the ore-unloading bridges which run on tracks placed on all four walls, spanning the four car tracks and the ore piles. The distance from the front wall to the inmost land wall is 419 ft.

Cleveland.—There is a full reinforced concrete ore-unloading dock at Cleveland, Ohio, completed in 1912 by the Pennsylvania Railroad. It consists of a reinforced concrete water-front wall, 985 ft. long, supported by a double row of pre-moulded concrete piles. The face wall is tied back by reinforced concrete tie-beams to another reinforced concrete wall supported by three rows of concrete piles about 81½ ft. in the rear. The tie-beams are 30 ft. apart, and rest on concrete piles 6 ft. apart. The space between the two walls, that is, under the concrete tie-beams, is back-filled with rip-rap, the whole mass being pumped full of sand. A concrete sheet-piling bulkhead under the front wall retains the fill in place. The dock has the usual wooden fender-pile protection. The tracks for four 17-ton Hewitt unloading machines run along the inner and outer walls. So far as the writer has been able to discover, this was the first concrete pile dock built on the Great Lakes.

Another ore-unloading dock at Cleveland consists of a reinforced wall of the same type as that at Detroit, the two structures having been designed by the same engineers. The cantilevered face of this wall is supported by brackets at intervals of 15 ft., with rear buttresses opposite them. The buttresses rest on a concrete base slab, the rear edge of which rests on a row of wooden sheet-piling, the base slab evidently being monolithic with the rest of the wall. The space between the sheet-piling and the wooden piles that support the concrete wall (11 ft.) is filled with slag. In general, the cross-section of the wall is dumb-bell shaped.

General.—In discussing the ore docks of the Great Lakes, it is necessary to consider the special and peculiar conditions under which they are operated, as they are designed to meet these conditions, and not *vice versa*. The larger portion of the freight carried on the Great Lakes is of heavy bulk form, *viz.*, ore and grain on the downward trip, coal on the upward trip. Thus ore and coal cargoes form the two heaviest items handled in bulk masses at the terminals. Nowhere in

the world is found such massive and modern machinery for the economical and expeditious handling of bulk freight as at the upper and lower ports of the Great Lakes. Consequently, such docks, wooden or otherwise, must not be judged by the type used along the seaboard of the Atlantic, the Gulf, or the Pacific.

It appears that the various types of reinforced concrete docks as worked out by the American engineers are far more numerous than in foreign practise, as is evident from the foregoing descriptions. Although the Atlantic Coast engineers seem to favor premoulded concrete piles, the Pacific Coast engineers apparently favor large concrete columns. Perhaps in time a typical American concrete dock will be designed or devised, as in the case of the long-standing type of wooden pile dock structures.

In the foregoing review of American reinforced concrete docks, an effort has been made to include each and every port wherein such types of docks exist, as well as to mention each and every dock already built, so far as the writer has been able to acquire sufficient information concerning them, in order that the exact situation as regards the development of reinforced concrete dock construction in America up to the present time may be known to all. If the writer has unintentionally omitted any such dock structure, he will be pleased to receive information relating to it.

The writer has endeavored to determine which was the first complete reinforced concrete dock constructed in the United States, but has been unable to do so. The concrete dock wall built at Chicago in 1898-99 appears to have been the first of its kind constructed on the Great Lakes. A study of the constructive dates of concrete pile or concrete column docks would indicate that such types began about 1905 or 1906. Still, it is not evident which was the first of such docks to come into existence, the whole development being a gradual evolution from a concrete-filled steel cylinder column, steel deck-beams, and concrete-slab type, as used in the Philippines by the United States Government in 1902.

Irrespective of the actual beginning of constructing reinforced concrete docks, it is generally conceded that Oscar F. Lackey, M. Am. Soc. C. E., Harbor Engineer of Baltimore, was among the first, if not the first, to blaze the way for the extensive use of reinforced concrete in dock construction in United States harbors.

PART III.—FAILURES OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SALT WATER ACTION.

In discussing reinforced concrete docks, the fact that there have been failures among them must not be overlooked. In Massachusetts waters, north of Cape Cod, a number of serious cases of deterioration of concrete have been caused by the disintegrating effects of sea water, wave action, and frost, especially in Boston Harbor, where nearly all the concrete structures standing in sea water have been affected badly between high and low tide, the most notable instance of which is the concrete pier at the Charleston Navy Yard. Although that part of the pier which is constantly submerged has given but little trouble, the part exposed alternately to the sea and air has been seriously affected, many large pieces having broken completely away, making it self-evident that some other agent than the chemical action between cement and sea water was at work.

As is well known, winter temperatures on the whole eastern front of the New England Coast run far below zero. In Boston Harbor 12° below zero is not uncommon. In the same way that hard earth and porous rocks are broken up by frost action, permeable concrete in freezing water will gradually be destroyed between wind and sea, as the water which gets into the concrete simply exercises its natural expanding function in freezing, which *a priori* is detrimental to the concrete structure. It is generally admitted that the exterior concrete in these Boston structures, especially in the Navy Yard pier, has failed almost entirely from the effect of the alternate freezing and thawing with each tide during the winter, due to permeable concrete.

A number of failures similar to that already cited have occurred in Boston, the disintegration taking place in all cases between low and high tides. In the case of the Dover Street draw-bridge pier, built in 1894, the disintegration had extended 1.4 ft. into the pier at the end of 17 years, the greatest damage being just below high-tide level. The pier was built of 1:2:5 concrete, with a 1-in. plastered mortar facing. English Portland cement was used throughout. Whether the 1-in. facing mortar was expected to act as a water-proof shield to the interior concrete is not apparent. Evidently, it did not act thus, as might have been expected.

As all the concrete in these disastrous cases seems to have been placed in the wet, that is, the sea water was allowed to come in con-

tact with the concrete before it had become thoroughly cured and hardened, such results are not to be wondered at, for one of the axioms of a successful use of concrete in sea water is that it must be kept from contact with sea water for such a period of time as to enable it to become thoroughly hardened, especially that part between tides in freezing climates.

In several cases in Boston Harbor where the concrete was placed inside of a coffer-dam, or used in the form of pre-moulded, driven, concrete piles, the concrete does not seem to have been affected as in the other cases cited. These successful cases go a long way toward substantiating the truism that concrete, to be used successfully in sea water, especially in freezing water, must be made impermeable in the process of making, with full consideration given to the brand of cement used, the mixture, the sand, and stone (or gravel), the skilled labor of placing, as well as keeping it from contact with sea water until it has set and hardened sufficiently. It is very apparent, from a study of the method used in placing the concrete in the disintegrated structures in Boston Harbor, that that method was far from possessing the essential features necessary for a successful solution of the problem, viewed in the light of present-day knowledge.

In comparison with these Boston failures, it is fitting to state that at Dundee, Scotland, where the climatic conditions are said to be worse than at Boston, and where there is a rise and fall of the tide of about 12 ft., the combined action of the sea, waves, and frost has had no ill effect on the concrete docks in that harbor, the concrete piles of which were allowed to harden for 30 days before being put in place.

Another noted case of the destruction of concrete by frost and sea action is the large concrete sea wall along the water front of Lynn, Mass.—a massive concrete sea wall exposed to the pounding of the winter storms and seas. The steps to the beach in the front of this wall were destroyed to such an extent as to be hardly recognizable as steps. It might be of interest to state that this wall and some of the damaged structures in Boston harbor have apparently been repaired effectively by the cement gun process.

In reviewing these failures in Boston and vicinity, it is well to consider the results obtained in using concrete in another port subject to freezing and ice conditions, *viz.*, New York Harbor. In addition to freezing conditions, New York Harbor has to contend with a

strong tidal effect, which results in large solid ice floes and fields of broken ice moving back and forth with a tide of considerable velocity, ice floes of such size coming down the Hudson as at times practically to compel abandonment of all transfer traffic in that river. This is an effect from which Boston docks are perhaps free, as no large rivers flow into that harbor, the Charles being kept under control by the so-called Charles River Dam.

In discussing this additional handicap and destructive force at work on New York City's 8¼ miles of concrete sea walls, some of which have been in existence for 41 years, Charles W. Staniford, M. Am. Soc. C. E., Chief Engineer of the New York Department of Docks and Ferries, states:

"Up to the present time [August, 1911], no disintegration has been discovered that can be attributed to the existence of the structure in salt water. The concrete itself is in an admirable state of preservation, absolutely hard, and is undergoing no regular process of disintegration." * * * "this sea-wall which has been under construction * * * for 41 years, is at the present time an excellent piece of work and is subject to the same climatic conditions as all cities on the Northern Atlantic Coast with the attending ice, cold and rain characteristic of this latitude."

In many instances, parts of this wall above low water are faced with granite blocks. This is a noted example of what can be expected in the way of using mass concrete in sea water if properly made, though perhaps some repair work has been necessary in order to maintain the excellent condition of the wall.

In some of the earlier sections of this wall the concrete was placed *"en-mass*-in-site", but, since 1876, most of the wall has been built by the concrete block method. Only under specially favorable conditions is it possible to place concrete successfully *in situ* under (sea) water, as it becomes disintegrated "through the chemical action of the sulphate of magnesia on fresh concrete or through the resulting porosity of concrete due to the impossibility of tamping under water",* the viscosity and weight of the mass not being sufficient to produce such a dense material as obtained in block work.

To discuss an opposite case in New York Harbor, *viz.*, Dry Dock No. 2, New York Navy Yard, originally built of timber in 1890, the

*This subject is discussed further by the writer in an article entitled "Chemistry of Salt Water Cement," *Metallurgical and Chemical Engineering*, January and February, 1914.

history of which it is not necessary to relate here: In 1900 this dock was rebuilt, concrete being used very extensively. During 1913 a large sum was expended in repairing and replacing the concrete altars and floors. As it has been stated that the difficulties of using concrete in sea water have been so great at this yard as to indicate that this is not a permanent material for use in sea water structures, it would be of deep interest to learn the facts as to the chemical composition of the cement used, of the sand and stone, as to the mixture thereof, and the precautions taken in mixing and placing; also as to whether the dock is kept flooded when not in use, especially during the winter. If, as has been stated, the concrete "has deteriorated and disintegrated to such an extent that it was possible to use a pick and shovel in removing it", it is apparent that it was lacking in one or more of the essential features that are deemed absolutely necessary for a successful use of concrete in sea water structures.

Whether any of the concrete pile docks on the Great Lakes have shown any signs of deterioration due to frost action, the writer does not know, but trusts that some facts covering this question will be brought out in the discussion. As the water level is practically the same all through the winter, only a very short length of the pile would be affected, and not some 10 ft., as in Boston Harbor.

One of the first concrete docks built in San Francisco is said to have failed in part due to poor construction. The early method of building the concrete columns of San Francisco concrete docks was to use a wooden cylinder, strongly built, as a column form, into which, it has been stated, the concrete was poured, apparently without any attempt to pump out the cylinder. As long as the wooden cylindrical forms remained in place around the supposedly concrete column, the dock was pronounced a success. When the teredo had finally destroyed the forms, the columns began to collapse and the dock became a pronounced failure, because, in pouring the concrete, the heavier material —the stone or gravel—settled first, then the sand, and finally the cement. The result was that throughout the length of the concrete columns there were alternate layers of uncemented stone and sand, with the cement in between the sand of one batch and the stone of the following one. Concrete can be and is successfully dropped through a height of 50 ft.—and even up to 100 ft. in one noted case

in Arizona—but, if the receptacle into which it is dropped is full of water, disaster alone awaits the unfortunate engineer.

In another of the San Francisco docks, where wooden piles supported the concrete columns, the concrete was not carried down below the mud line a sufficient distance to prevent the teredo from destroying the piles below the concrete.

The question has been raised: Has any deterioration taken place in concrete structures standing in sea water in the harbors of the Southern States, where frost action is unknown. The most prominent concrete structure thus situated is the famous viaduct across the Florida Keys, built of Alsen cement, imported from Germany. It is possible that some of the members of this Society are in a position to give complete information regarding the action of salt water and the waves of the Gulf on this structure.

In order to guard against the disintegration (irrespective of its cause) of mass concrete placed *in situ* above low water, or to repair any damage that has been done, besides the cement gun process, various methods have been used, all based on the fundamental principle of using an impermeable material for the facing of the structure. Below low water, properly made block work has given most satisfactory results. Carefully made, fully cured pre-moulded concrete piles seem to resist the action of the sea and frost successfully. In Holland, hard, impermeable brick have been used to prevent any further damage to one of the breakwaters above low water. In England, the upper parts of massive breakwaters are mostly faced with granite or some other hard suitable stone. In Nova Scotia, both brick and pre-moulded blocks of concrete of small size were placed on the face of a concrete sea wall after the disintegrated concrete had been removed. A still more recent device is the use of hollow, vitrified, salt-glazed tile blocks filled with concrete after being put in place. Experiments thus far seem to have proved that:

"Vitrified salt-glazed tile is impervious to any deteriorating action of sea water, and has an effective structure against the battering of ice; it is so dense as to preclude the possibility of any water entering and freezing in it to the consequent destruction of the tile."

Though oiled concrete is being used as a water-proof material in certain cases, it is possible that the refuse, oil, gases, etc., discharged from certain classes of buildings, etc., might have a destructive effect

on the concrete foundation piles or other parts of the building, especially in sea water heavily charged with sewage. It is a well-known fact that concrete sewers will not perform their duty properly for any length of time unless they have a brick lining invert, over which flows the heavy sludge. In time of flood the surface water is so great as to dilute the sewage and prevent injurious effects. The writer would be pleased to hear opinions on this point, as it is possible that a destructive effect might have been caused by sewage in connection with one of the most seriously affected cases in Boston.

Although poor results seem to have attended quite a number of the reinforced concrete structures standing in sea water in America, the opposite appears to have been true in foreign countries. Still, a few failures are on record as having occurred in England and Germany, due mostly to permeable concrete.

Part IV.—Construction and Maintenance Cost of Reinforced Concrete Docks.

Cost of Construction.—To attempt any discussion of costs is always attended with danger to the one who does so, especially when such figures are "published cost". To lay the foundations for a discussion of this side of the question, the data in Table 2 have been collected from various publications, their real worth depending on the reliability of the published figures. The data cover some of the docks mentioned in Parts I and II.

The first cost of the various reinforced concrete dock structures at Port Talbot is stated to have been "no more than if they had been built of wood." Such a statement would not perhaps be true in the United States, on account of the large forests in this country.

Maintenance Cost.—A few figures covering the cost of maintenance of reinforced concrete docks in England are noted. In addition to what has already been said on the maintenance question, the cold storage wharf at Southampton is reported to have cost nothing to date for maintenance. On the other hand, the widened dock and the coal jetties are said to have shown considerable deterioration due to rusting of the steel, it having been improperly placed in these structures. It has been stated authoritatively that "while six to seven years is perhaps a rather short time in which to form any definite conclusions, the maintenance cost [of the above described Port Talbot docks] has been practically nothing."

The annual repair charges on the Purfleet coaling jetty (exclusive of the damage done at the time of the collision) for the first 9 years of its existence are stated to have been but $50 per annum, which, based on its cost, $60 000, is less than one-tenth of 1 per cent.

TABLE 2.—COST OF CONCRETE DOCKS.

Location.	Type.	Cost per square foot.
Pier No. 8, Puget Sound Navy Yard...	Concrete columns. Steel deck-beams. Concrete deck-slab.	$3.11
Naval Station, Philippines.....	Concrete columns. Steel deck-beams. Concrete deck-slab.	2.60
Balboa, Panama Canal.....	Concrete columns. Concrete beams. Concrete deck-slab.	3.28
Oakland, Cal.....	Concrete piles. Concrete beams. Concrete deck-slab.	3.27
Brunswick, Ga.....	Concrete piles. Wooden deck system.	1.40
Charleston Navy Yard, S. C.....	Concrete piles. Wooden decking.	2.60
United Fruit Company, Panama.....	Concrete-protected wooden piles. Concrete deck-beams. Concrete slab.	2.13
Brooklyn, average of two docks.....	Wooden piles. Wooden caps. Concrete deck-slab.	0.90

Table 3, from data published by the Chief Engineer of the Mersey Dock and Harbor Board in 1910, gives some very interesting results as respects the cost of annual repairs to six of the reinforced concrete docks at Southampton.

TABLE 3.—ANNUAL REPAIRS, SOUTHAMPTON DOCKS.

Dock.	Erected.	Cost.	Cost of repairs to date.	Average per year.	Annual percentage, based on original cost.	Remarks.
A	1899–1900	$24 000	$800	$80	0.0033	Deck for Cattle Wharf, Prince's Jetty (wood piles).
B	1904–06	63 500	375	75	0.0012	Floor or deck on Hennebique piles, Prince's Dock, West Quay.
C	1900	13 800	80	8	0.0005	Floor for wharf, Coburg Quay.
D	1901	3 700	25	3	0.0008	Floor for wharf, etc., Brunswick Half Tide Dock.
E	1908	17 500	.....	....	.....	Floor for wharf, etc., North Quay, Brocklebank Dock.
F	1908	163 500	.....	....	.....	Treble-story shed, South Quay, Sandon Dock.

Dock *A*. Subject to the effect of moist air arising from water below it.

Dock *B*. Subject to effect of moist air. Piles more or less submerged, according to water level.

Dock *C*. Complete reinforced shed and pile foundation: not sufficient time to form any conclusion.

If the results in Table 3 are true, it is no wonder that the English engineers report that their reinforced concrete docks cost nothing for annual repairs.

Part V.—General Considerations.

Opinions of Foreign Experts.—Although failures have accompanied the use of concrete in sea water and in the construction of reinforced concrete docks, it must be admitted that, if the Engineering Profession did not meet with a failure now and then, it would never acquire anything new, as it is through failures that it gains the most vital knowledge of engineering.

(*a*).—In discussing the question of concrete docks, a prominent New York engineer has stated that, of the large number of concrete docks which have come under his observation, the majority have been a success, though here and there he reports a failure due to poor construction and material, and not to defects in the design. It is authentically stated that the reinforced concrete docks at Southampton have shown no deterioration due to salt water action, except at the Southampton coal jetty. The engineers of the Liverpool Docks have been using concrete in connection with their work since 1872, apparently with great success. It has been stated on the best authority that in England the alternation of "dryness and wetness and fluctuations in temperature" does not appear to have affected reinforced concrete sea water structures adversely.

Mr. Henry Hunter, Chief Engineer of the Manchester Canal, England, states that "the concrete in the concrete lock built at Eastham is in better condition at the present date than the day it was deposited", and adds that, covering an experience of more than 30 years of placing concrete in salt water, he has known no failures in such work where the concrete has been properly mixed and deposited.

In discussing the concrete docks of the Port Talbot Dock Company, Mr. William Cleaver has stated that:

"While reinforced concrete requires extreme care, both in the choice of material and in the supervision of the workmanship, the results justify the extensive adoption of the material for dock work."

The exceptionally experienced dock engineer, Mr. Francis E. Wentworth-Shields, of the London and Southwestern Railway Company, has stated that "if great care is exercised in making and placing concrete,

an impermeable material will be obtained which can withstand the action of salt water." Mr. Shields has also said, "while many engineers were nervous about the life of reinforced concrete (for sea structures), he had observed that maintenance engineers were not so nervous as construction engineers." He is inclined to feel that there is nothing special to be feared respecting the life of reinforced concrete when used in marine work. Although, under certain circumstances, it is likely to deteriorate, he does not think it will do so from simply standing in sea water. He says that though in some cases deterioration has taken place at Southampton above low water, it has not done so below low water; that a 10-year-old reinforced concrete structure standing in sea water at Southampton is in perfect condition at the present date; and that, during the whole experience at Southampton, sea water does not seem to have produced any chemical or other deleterious action on the concrete.

Experiments by Mr. Baldwin-Weisman, in 1907, in England, on the permeability of concrete, show that, if it is well made, it is one of the most water-tight materials known, and that it rapidly becomes less and less porous when water is forced through it.

Mr. V. De Blocq von Keuffeler, in summing up the experience in using concrete for salt water structures in Holland, says:

"A suitable mixture, very carefully manufactured, the use of a good brand of cement with trass, and setting in a moist atmosphere, are the most efficient means of ensuring the preservation of reinforced concrete in sea water."

Mr. I. Ho, one of Japan's expert harbor engineers, who used more than 1 200 mass-concrete blocks in one instance, none of which during a period of 10 years has shown the slightest signs of failure, states, "that whereas a good and proper cement is of consideration, the most important factor is the mode of fabrication."

In reviewing the successful experiences of some of England's leading authorities, the question of the chemical composition of the cement used does not appear to be given. Such information would be of great value, in order that engineers may know whether they use a cement especially manufactured for sea water concrete or simply the ordinary Portland cement, with or without puzzolana, trass, etc.

(*b*).—In discussing the deterioration of steel in reinforced concrete, by the action of sea water on ferro-concrete, provided the latter is properly made, Mr. C. S. Meiks, a prominent concrete engineer of England, says that such deterioration "is a negligible quantity." In

support of this contention he cites the experience at Southampton, stating "that the exposed steelwork on a pile end that had been in the sea for 8 years was much corroded", whereas the bars in the body of the concrete, on being cut open, were found to be quite free from any rust and as fresh as the day they were put into the pile.

In connection with the building of some of the earlier concrete dock structures at Southampton, it appears that parts or the whole of piles not used were allowed to remain on the beach or shore, exposed to sea water, for some 7 years. At the end of this period the exposed steel had been badly rusted and deteriorated, whereas the part which was embedded in concrete was found to be in fine condition, practically as good as the day it was placed in the concrete. Still, concrete piles lying on the beach are not in the same position as concrete piles subjected to shocks in a dock.

Though the first jetties, built 11 years ago in Southampton water, are in excellent condition at present, the steel in another jetty at the same location has deteriorated, due to electrolytic action.

It is generally accepted by all English authorities that no deterioration takes place in steel when well embedded in the concrete.

(*c*).—In speaking of reinforced concrete when used in marine work, Mr. Wentworth-Shields says "it will stand a wonderful amount of shock, and bending due to shocks, if a wooden fender is interposed." At a more recent date, Mr. Shields remarked: "On the other hand, reinforced concrete would not bear being knocked about by heavy ships, and where a structure was subjected to severe blows of that sort it was not easy to find anything better than timber, * * * but, when used at the right time and in the right place, reinforced concrete was a valuable material to dock engineers." From the leading position Mr. Shields occupies among the dock engineers of England, it would be of interest to know just what distinction there is between "a wonderful amount of shock" and "subjected to severe blows."

As an axiom: whatever system or design is adopted for a reinforced concrete dock, in no manner whatsoever should a vessel be allowed to rub against the main piling of the dock. The dock should always be protected by a system of fender-piles.

(*d*).—As respects the resistance of a concrete pier or dock, when under such treatment as the Purfleet pier was at the time it was rammed, in 1904, Mr. Meiks, engineer in charge of its construction,

states that "the vibration was so great at the time of the collision that they thought the entire pier would collapse, but that its elasticity was most satisfactory, due no doubt to its horizontal concrete decking." Mr. Meiks also says that the vibration of a concrete pier supported on piles is nearly as great as in a pier made of timber piling; but this has no particular effect on the structure, judging by the experience gained with the Purfleet pier at the time it was rammed.

In speaking of the Port Talbot docks, Mr. A. E. Carey has stated:

"If the structure [reinforced concrete dock] was properly designed and built, its stability and life were assured, the only serious drawback being the difficulty of repairing damage due to collision."

But how often do collisions happen?

The Chief Engineer of the Port of London, Mr. Bryson Cunningham, recently stated that the art of building reinforced concrete docks in England has attained a degree of perfection greatly in advance of early experimental work. If their early experimental docks are still doing good service, will not their more recent docks become structures of an engineering and commercial success, thus justifying the American engineer in recommending concrete docks as long as they are built in a manner to guarantee impermeability and non-deterioration of the concrete?

Concrete Breakwaters.—As the application of reinforced concrete to dock construction has been developed almost entirely since 1900, some of the most conservative engineers may not feel that sufficient time has elapsed to judge correctly as to the merits of using cement and placing concrete structures in sea water, and as to the advisability of adopting reinforced concrete as a coming type of dock structure. As respects the first point, the prolonged and successful use of mass concrete by foreign countries in breakwater, tidal, and graving dock work would in itself appear to be sufficient answer to all such skepticism.

Though an extensive treatise might be written on concrete breakwaters and their construction, including shore protection, those phases of the use of concrete in sea water are supplementary to the principal subject of this paper. There is apparently hardly a leading seaport or a maritime nation outside of the United States that has not made a wide, extensive and successful use of mass and reinforced concrete in the development of harbor and shipping facilities.

Surprising as it may seem, a number of large concrete breakwaters

have been in existence in Japan for more than 18 years. As a matter of fact, the use of concrete in the harbor work of Japan is far in advance of American practice, being apparently on the same high level as in England and other European countries.

Mass concrete has been used very extensively in Belgium and Holland for sea water structures for years, and has given the best of results. In fact, some of the concrete sea walls in the latter country, built in 1867-77 and earlier, are so old as to be called ancient, and have as yet shown no signs of being affected by the action of sea water. Perhaps such a statement needs to be qualified, because some of the principal harbors of Holland are some distance from the sea, and are in fresh or brackish water.

Concrete was used by the Romans and Carthagenians in ancient times. Though it fell into disuse for many centuries, it came into use again in 1840-50. To this day, sea walls built of puzzolana and lime cement by the Romans are in existence in Italy.

The Italian engineers report that Portland cement concrete with an addition of one-eighth to one-tenth by volume of puzzolana gave no signs of disintegrating in salt water, even after an exposure of 30 years in the harbors of Genoa, Civita Vecchia, Naples, etc.

At several places on the Italian coast concrete-faced breakwaters have been in existence since 1880, in most exposed positions, costing but little for repairs and maintenance, though subject to the high seas and heavy blows of the Mediterranean.

English engineers were using mass concrete in their tide locks and in the construction of massive breakwaters along the coast of England as far back as 1871, if not earlier. The fact that they have continued to use it more extensively each year, even to building vast reinforced concrete structures standing in sea water during the past 15 years, would appear, in spite of some failures, to be sufficient answer to any doubts the American engineer may entertain on the subject.

At Colombo, India, a concrete breakwater, finished in 1885, showed no failures above or below water at the end of 22 years. As stated above, some of the massive concrete breakwaters of the world have a hard stone facing, or are built of concrete blocks, with or without a stone facing.

The reasons that enable the foreign engineers to accomplish such lasting results with concrete is no doubt due to the fact that they, to-

gether with the foreign chemists and cement manufacturers, long ago learned the secret and acquired the art of manufacturing and using concrete in sea water structures. Though the American engineer excels the foreign engineer in certain lines of his profession, it must be admitted that, so far as using cement, and hence concrete, in sea water structures, the engineers of the leading European countries and of certain parts of South America are many years in advance. The American cement manufacturer, the chemist, and the harbor development engineer cannot long remain in such a position without reflecting on their ability as experts in their respective lines of work in the minds of their foreign contemporaries.

PART VI.—CONCLUSION.

In view of the marked success obtained by foreign engineers in the use of concrete for sea water structures, when the execution of such undertakings has been placed in the hands of intelligent, skilful, and experienced men, the American engineer who denies the possibility of making a successful use of concrete for structures standing in sea water puts himself in a questionable position. He thereby confesses either his lack of a world-wide knowledge on the subject, or his inability to carry out properly such classes of construction work to the same successful conclusions as his foreign contemporaries have been doing for years past. Such a confession would seem to indicate a lack of foresight and ultra-conservatism as respects the use of cement subject to sea water conditions, on the part of the American cement manufacturer, chemist, and concrete engineer.

The American engineer who assumes a skeptical attitude toward the practicability and commercial success of reinforced concrete docks has standing before him as silent testimony of their worth and practicability such a vast number of foreign reinforced concrete docks—several about 20 years old and yet in excellent condition, with still more massive structures being built each year, some of them costing less than wooden docks, if reports are true, and most of them saving their owners large sums annually on account of their low cost of maintenance and repairs, with no rebuilding, as with our 15-year creosoted wooden pile docks—that the grounds on which he stands become somewhat untenable. It is true that all American cements are not as yet wholly suitable for sea water purposes, and perhaps equally true that each and every American concrete engineer has not hitherto insisted

on the proper placing of concrete in salt water structures, not fully realizing the importance of the fundamental principles of the use of concrete in sea water, due to the hitherto limited call for such types of structures in America, lumber having been so plentiful and cheap.

Though reinforced concrete docks have been in existence for more than 15 years, and operated successfully, the same old theorems are still put forth in opposition to them and to the practical experience gained during this period. Possibly these same theories will continue to be advanced against the use of concrete in dock work and other sea structures by the most conservative of our leading engineers, though others, guided and profiting by the experience already gained in such uses of concrete, will continue to expend large sums of money in the further development of such structures.

From a prolonged study of reinforced concrete dock construction, as carried out in foreign countries, it would appear that, in spite of early doubts and skepticism, the success in the use of reinforced concrete in dock work obtained by foreign engineers has swept away all such doubts and skepticism. If these are not facts, why are foreign countries and certain ports in America, including the United States Government, expending vast sums of money in building reinforced concrete docks and in other uses of concrete in harbor development and sea protection work; all "in spite of prejudices which leading engineers [psychologically] have against any new type of construction."

Although the average American contractor may look upon concrete as just so much cement, sand, and stone, or gravel, to be thrown together and dumped into the forms in the quickest possible time, without any regard for the fundamental principles underlying reinforced concrete construction, a commercial proposition purely, such an application of reinforced concrete to dock work will most certainly spell disaster long before the structure is completed.

In spite of its apparent simplicity on dry land, the use of reinforced concrete in dock work calls for more than mere brawn and muscle. It is a class of construction work especially adapted to the broad knowledge, experience, and deep study of the trained engineer in association with an organization well skilled in the handling of concrete in sea water structures—"a field of engineering in which reinforced concrete will prove to be the most permanent and economical, as it has in the building of bridges, etc."

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FLOOD FLOWS.
Discussion.*

By Messrs. G. B. Pillsbury and Weston E. Fuller.†

Mr. Pillsbury.

G. B. Pillsbury, Assoc. M. Am. Soc. C. E. (by letter).—Accepting the premise that the magnitude of a flood is due to the combination of an unlimited number of accidental causes, the probability of the occurrence of a flood exceeding a given magnitude should follow the well-established laws of probability as deduced in the theory of errors of observations. Under these laws the probability of an annual flood less than a given value, Q', is given by the formula:

$$P = \frac{h}{\sqrt{\pi}} \int_0^k e^{-h^2 x^2} \, dx \ldots\ldots\ldots\

Mr. Pillsbury.

The usual probability tables give the value of the function, $\frac{2}{\sqrt{\pi}}\int_0^t e^{-t^2}\,dt$, from which the function, $\frac{1}{\sqrt{\pi}}\int_t^\infty e^{-t^2}\,dt$, in Equation 2 can be computed from the relation,

$$\frac{1}{\sqrt{\pi}}\int_t^\infty e^{-t^2}\,dt = \frac{1}{2}\left(1-\frac{2}{\sqrt{\pi}}\int_0^t e^{-t^2}\,dt\right).$$

The author, however, has taken the average value of the annual floods equalling or exceeding the given value as his measure of the expected flood, and not the flood of magnitude which would be derived from Equation 2.

The mathematical expression for this average or mean of floods exceeding Q' is

$$Q = Q\,(\text{Ave.}) + \frac{\frac{h}{\sqrt{\pi}}\int_k^\infty x\,e^{-h^2x^2}\,dx}{\frac{h}{\sqrt{\pi}}\int_k^\infty e^{-h^2x^2}\,dx}$$

$$= Q\,(\text{Ave.}) + \frac{1}{h}\,\frac{\frac{1}{2\sqrt{\pi}}e^{-t^2}}{\frac{1}{\sqrt{\pi}}\int_t^\infty e^{-t^2}\,dt} \quad \ldots\ldots(3)$$

$$= Q\,(\text{Ave.}) + \frac{1}{h}\,M \quad \ldots\ldots(4)$$

It is obvious that this mean value is somewhat greater than its inferior limit.

Table 39 shows the relation between the time, T, considered (or the reciprocal of P in Equation 2), and the value of M (Equation 4).

TABLE 39.

t	$T = \frac{1}{P}$	M
0.0	2.	0.564
0.2	2.57	0.697
0.4	3.50	0.841
0.6	5.05	0.994
0.8	7.77	1.155
1.0	12.72	1.32
1.2	22.32	1.49
1.4	42.02	1.67
1.6	84.88	1.85
1.8	185.2	2.05
2.0	434.8	2.25
2.2	1 000.	2.34

Mr. Pillsbury.

Taking the floods of Tohickon Creek, as given by the author in Table 4, the computation of the mathematical values of average maximum flood, under the theory of probabilities, is shown in Table 40.

TABLE 40.

Date.	Annual flood.	Ratio to average.	V	V^2
1884	4 379	1.06	+ 0.06	0.0036
5	3 664	0.89	− 0.11	0.0121
6	5 359	1.30	+ 0.30	0.0900
7	2 544	0.62	− 0.38	0.1444
8	3 493	0.85	− 0.15	0.0225
9	4 714	1.15	+ 0.15	0.0225
1890	2 942	0.71	− 0.29	0.0841
1	2 858	0.69	− 0.31	0.0961
2	3 158	0.76	− 0.24	0.0576
3	2 994	0.73	− 0.27	0.0729
4	8 650	2.10	+ 1.10	1.2100
5	3 857	0.93	− 0.07	0.0049
6	6 515	1.59	+ 0.59	0.3461
7	3 683	0.89	− 0.11	0.0121
8	4 160	1.01	+ 0.01	0.0001
9	3 222	0.78	− 0.22	0.0484
1901	4 089	0.99	− 0.01	0.0001
2	5 958	1.45	+ 0.45	0.2025
3	4 968	1.21	+ 0.21	0.0441
4	4 395	1.06	+ 0.06	0.0036
5	4 175	1.01	+ 0.01	0.0001
6	3 200	0.78	− 0.22	0.0484
7	4 120	1.00	0	0
8	2 770	0.67	− 0.33	0.1089
9	3 050	0.74	− 0.26	0.0676
Average = 4 117		$\Sigma v^2 = 2.7027$		$h = 2.11$.

The probable average maximum floods, according to the theory of probabilities, and the observed averages as determined by the author, are plotted on Fig. 14.

Table 41 and Fig. 15 show the floods of the Connecticut River at Hartford for 70 years, beginning 1843. The observed average maximum floods shown on the figure are computed in Table 42.

The floods of the Allegheny River at Kittanning for a 41-year period are tabulated in Mr. Knowles' discussion, Table 33. The value of h, as derived from these data is 2.03. The curve of probable average maxima, the author's curve, and the observed values, are plotted in Fig. 16.

A very cursory examination of the figures shows that, for individual rivers, the observed average maxima differ widely, both from the author's formula and from their theoretical values under the theory of probability. This, indeed, is not extraordinary. The highest of the plotted observed figures is but a single flood, and not an average. The first few of the succeeding observed averages are based on very limited data, and may be expected to differ from their theoretical values in the long run. In the case of Tohickon Creek, the extraordinarily

Mr. Pillsbury.

TABLE 41.—YEARLY FRESHETS IN THE CONNECTICUT RIVER AT HARTFORD.

Year.	Maximum gauge.	Max. yearly flood.	Ratio.	V	V^2
1843	27.2	175 000	1.55	+ 0.55	0.3025
4	19.5	97 000	0.86	− 0.14	0.0196
5	19.0	93 000	0.83	− 0.17	0.0289
6	18.7	90 000	0.80	− 0.20	0.0400
7	21.0	110 000	0.98	− 0.02	0.0004
8	15.5	68 000	0.60	− 0.40	0.1600
9	17.5	81 000	0.72	− 0.28	0.0784
1850	20.7	107 000	0.95	− 0.05	0.0025
1	14.5	61 000	0 54	− 0.46	0.2116
2	23.1	130 000	1.15	+ 0.15	0.0225
3	20.5	106 000	0.94	− 0.06	0.0036
4	29.8	205 000	1.82	+ 0.82	0.6724
5	15.0	64 000	0.57	− 0.43	0.1849
6	23.3	132 000	1.17	+ 0.17	0.0289
7	19.5	97 000	0.86	− 0.14	0.0196
8	12.2	48 000	0.43	− 0.57	0.3249
9	26.4	166 000	1.47	+ 0.47	0.2209
1860	16.0	71 000	0.63	− 0.37	0.1369
1	21.5	115 000	1.02	+ 0.02	0.0004
2	28.7	192 000	1.70	+ 0.70	0.4900
3	15.0	64 000	0.57	− 0.43	0.1849
4	17.2	79 000	0.70	− 0.30	0.0900
5	24.7	147 000	1.30	+ 0.30	0.0900
6	20.5	106 000	0.94	− 0.06	0.0036
7	20.0	101 000	0.90	− 0.10	0.0100
8	21.5	115 000	1.02	+ 0.02	0.0004
9	26.5	167 000	1.48	+ 0.48	0.2304
1870	25.3	154 000	1.37	+ 0.37	0.1369
1	18.5	89 000	0.79	− 0.21	0.0441
2	19.7	99 000	0.88	− 0.12	0.0144
3	20.9	109 000	0.97	− 0.03	0.0009
4	23.8	137 000	1.22	+ 0.22	0.0484
5	18.4	90 000	0.80	− 0.20	0.0400
6	21.9	121 000	1.07	+ 0.07	0.0049
7	22.8	130 000	1.15	+ 0.15	0.0225
8	23.9	142 000	1.26	+ 0.26	0.0676
9	21.4	116 000	1.03	+ 0.03	0.0009
1880	15.0	66 000	0.59	− 0.41	0.1681
1	16.4	75 000	0.67	− 0.33	0.1089
2	14.7	64 000	0.57	− 0.43	0.1849
3	20.5	108 000	0.96	− 0.04	0.0016
4	21.9	121 000	1.07	+ 0.07	0.0049
5	18.1	88 000	0.78	− 0.22	0.0484
6	21.7	119 000	1.06	+ 0.06	0.0036
7	22.5	127 000	1.13	+ 0.13	0.0169
8	19.4	98 000	0.87	− 0.13	0.0169
9	15.6	70 000	0.62	− 0.38	0.1444
1890	16.0	73 000	0.65	− 0.35	0.1225
1	19.8	102 000	0.90	− 0.10	0.0100
2	....	.....	...	.....	.....
3	24.0	143 000	1.27	+ 0.27	0.0729
4	13.8	59 000	0.52	− 0.48	0.2304
5	25.7	161 000	1.43	+ 0.43	0.1849
6	26.5	170 000	1.51	+ 0.51	0.2601
7	20.8	111 000	0.99	− 0.01	0.0001
8	21.2	114 000	1.01	+ 0.01	0.0001
9	22.0	122 000	1.08	+ 0.08	0.0064
1900	23.4	136 000	1.21	+ 0.21	0.0441
1	26.4	169 000	1.50	+ 0.50	0.2500
2	25.5	159 000	1.41	+ 0.41	0.1681
3	23.3	135 000	1.20	+ 0.20	0.0400
4	19.2	97 000	0.86	− 0.14	0.0196
5	24.0	143 000	1.27	+ 0.27	0.0729
6	19.8	102 000	0.90	− 0.10	0.0100
7	20.7	110 000	0.98	− 0.02	0.0004
8	18.1	88 000	0.78	− 0.22	0.0484
9	24.7	150 000	1.33	+ 0.33	0.1089
1910	19.4	99 000	0.88	− 0.12	0.0144
1	14.8	65 000	0.58	− 0.42	0.1764
2	20.5	108 000	0.96	− 0.04	0.0016
3	26.0	165 000	1.46	+ 0.46	0.2116

Average = 112 700 $\Sigma v^2 = 6.6912$ $h = 2.27$.

Fig. 14.

Mr. Pillsbury.

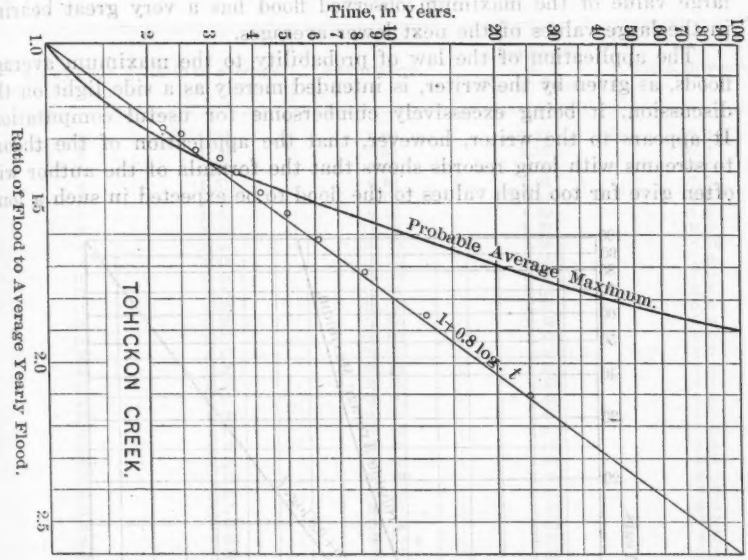

FIG. 14.

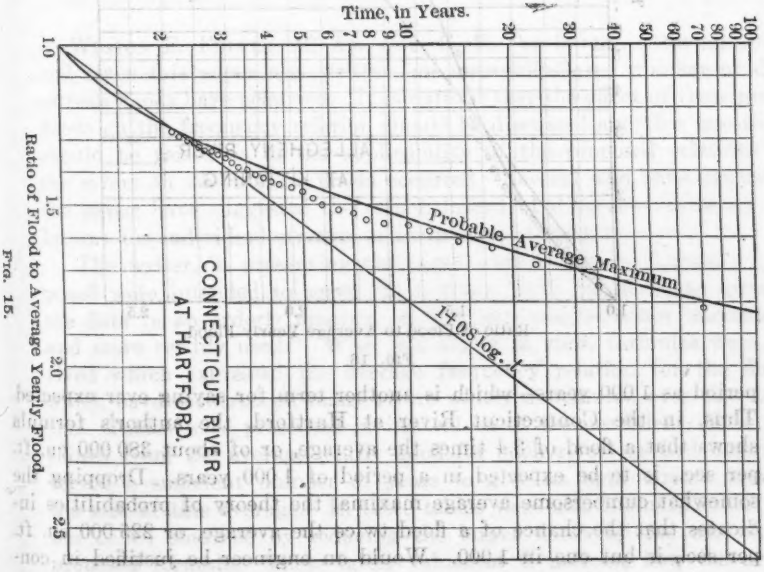

FIG. 15.

Mr. Pillsbury.

large value of the maximum observed flood has a very great bearing in the large values of the next lower averages.

The application of the law of probability to the maximum average floods, as given by the writer, is intended merely as a side light on the discussion, it being excessively cumbersome for useful computation. It appears to the writer, however, that the application of the theory to streams with long records shows that the formula of the author will often give far too high values to the flood to be expected in such a long

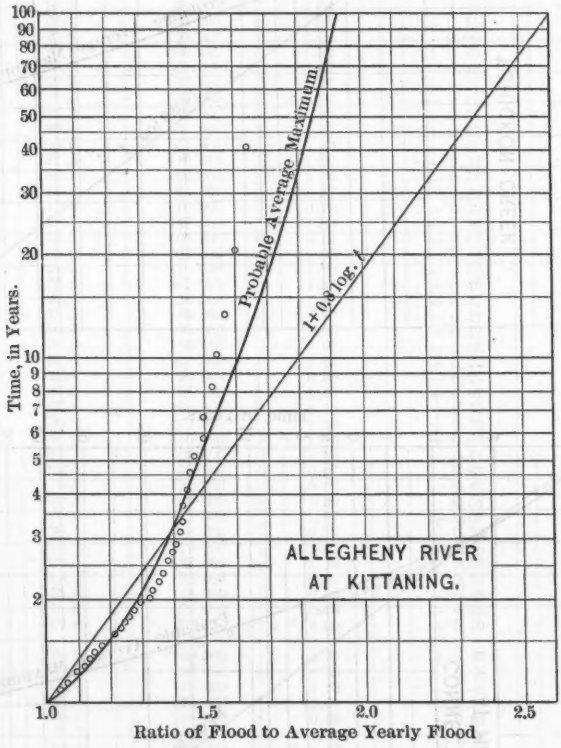

FIG. 16.

period as 1 000 years—which is another term for saying ever expected. Thus, in the Connecticut River at Hartford, the author's formula shows that a flood of 3.4 times the average, or of about 380 000 cu. ft. per sec., is to be expected in a period of 1 000 years. Dropping the somewhat cumbersome average maxima, the theory of probabilities indicates that the chance of a flood twice the average, or 225 000 cu. ft. per sec., is but one in 1 000. Would an engineer be justified in constructing works to care for more than, say, 250 000 cu. ft. per sec.?

TABLE 42.

Mr. Pillsbury.

No. of flood.	Ratio to average.	Summation.	Summation ÷ No. of flood.	Time, in years.
1	1.82	1.82	1.82	70.0
2	1.70	3.52	1.76	35.0
3	1.55	5.07	1.69	23.3
4	1.51	6.58	1.64	17.5
5	1.50	8.08	1.61	14.0
6	1.48	9.56	1.59	11.6
7	1.47	11.03	1.57	10.0
8	1.46	12.49	1.56	8.75
9	1.43	13.92	1.55	7.8
10	1.41	15.33	1.53	7.0
11	1.37	16.70	1.52	6.4
12	1.33	18.03	1.51	5.8
13	1.30	19.33	1.49	5.4
14	1.27	20.60	1.47	5.0
15	1.27	21.87	1.46	4.7
16	1.26	23.13	1.44	4.4
17	1.22	24.35	1.43	4.1
18	1.21	25.56	1.42	3.9
19	1.20	26.76	1.41	3.7
20	1.17	27.93	1.40	3.5
21	1.15	29.08	1.39	3.3
22	1.15	30.23	1.38	3.2
23	1.13	31.36	1.36	3.0
24	1.08	32.44	1.35	2.9
25	1.07	33.51	1.34	2.8
26	1.07	34.58	1.33	2.7
27	1.06	35.64	1.32	2.6
28	1.03	36.67	1.31	2.5
29	1.02	37.69	1.30	2.4
30	1.02	38.71	1.29	2.3
31	1.01	39.72	1.28	2.26

Mr. Fuller.

WESTON E. FULLER, M. AM. SOC. C. E. (by letter).—During 1913, and since this paper was written, an unusually large number of disastrous floods have occurred. It is natural that the effect of these great floods on the frequency relation should be discussed and that questions should be raised as to the applicability of the proposed relations to the rivers on which these floods occurred. Several who have discussed the paper have suggested the establishment of different frequency relations for individual streams or for groups of streams.

The writer, in presenting the paper, stated that the formulas proposed were intended to serve "as a frame work on which to arrange the data in an orderly manner, so that they can be better understood and more readily used." With this object in view, formulas were derived which expressed the average frequency relation for the floods which have occurred on many rivers widely distributed over the country. Tables were presented giving values of the coefficient, C, as obtained from the average yearly flood for such rivers as had been observed a sufficient number of years to give an approximate idea of the size of this average. In these tables were also included other values of C, obtained by reducing the larger floods by the use of the proposed formula. These latter values of C indicate how the actual

Mr. Fuller.

floods which have occurred on the rivers have agreed with the proposed formula. The close agreement, for most of the rivers, of these two sets of coefficients indicates that the relation is a general one. During the course of the study on which the paper was based, plottings were made for many individual streams and for groups of streams in different sections of the country. The effect of the few larger floods on the plottings for individual streams was so great that the writer concluded that the relation indicated by them was less accurate than the average relation. Plottings for different sections of the country varied to some extent. If streams in partly arid sections are excluded, the variation of the coefficient of log. T in the formula is from 0.7 to 1. In the extreme cases the number of streams and the length of the records were too short to furnish proof sufficiently strong to justify any change from the general relation. That different frequency relations do exist for individual streams and for different sections of the country is probable, and, as more data become available, such relations may be established.

Great floods which have occurred on the streams in Ohio since the paper was written indicate either that in this section of the country great floods occur more frequently than in other sections, or that these floods were very extraordinary ones. It is unfortunate that so few data are available for floods on the rivers of Ohio, Indiana, Illinois, and, in fact, the streams in all the States along the Mississippi Valley. Floods on streams in adjoining sections indicate that the relation is similar to the average one proposed in the paper. The floods of 1913 in Ohio were caused by rainfall of extraordinary intensity, considering the large area covered, occurring under conditions favorable for great floods. That these conditions were extraordinary is certain, but whether such conditions occur more frequently in this section than elsewhere only the future will show.

Floods of 1913.—Although data are not yet available for all the great floods which occurred during 1913, records of those on some rivers in Ohio and New York have been published. Among those of particular interest are the floods on the Miami River at Dayton, Ohio, the Scioto and Olentangy Rivers at Columbus, Ohio, and the Hudson River at Mechanicsville, N. Y. The data for these rivers, revised to include the records of 1913, are given in Table 43.

The value given for the ratio between the flood of 1913 and the average flood for the streams at Columbus corresponds to a value of T equal to about 1 400. As there are more than 1 400 records of floods on different rivers for which the ratios to the average flood are now available, the occasional occurrence of such a flood is not surprising.

For the Miami River at Dayton, Mr. Morgan expresses the opinion that the coefficient, C, in the writer's formula should not exceed 50. If this were the proper value for C, the 1913 flood on the Miami

TABLE 43.

Mr. Fuller.

Name of stream.	Location of station.	Catchment area, in square miles.	Flood Flows, in Cubic Feet per Second.		Period of observation, in years.	Ratio of maximum flood to average flood.	Probable value of C.
			Average yearly.	Floods of 1913.			
Upper Scioto...	Water-works dam...	1 032	19 300	68 000	16	3.52	75
Olentangy.....	Columbus, Ohio.....	520	14 500	51 000	16	3.52	97
Lower Scioto...	Columbus, Ohio.....	1 570	33 800	119 000	16	3.52	94
Miami	Dayton, Ohio.....	2 450	50 000*	246 000*	21	4.92	90
Hudson.....	Mechanicsville, N. Y.	4 500	44 500	108 000	23	2.42	53

* Represents rate at maximum stage.

The data for the floods at Columbus are from the "Report on Flood Protection for the City of Columbus" by John W. Alvord and C. B. Burdick, Members, Am. Soc. C. E.

The data for the maximum flood at Dayton are from the discussion by Arthur E. Morgan, M. Am. Soc. C. E.

The data for the maximum flood on the Hudson are from *Engineering Record*, April 12th, 1913, "Effect of Recent Floods on New York Streams" by R. E. Horton, M. Am. Soc. C. E.

River would be relatively very greatly in excess of any flood which we have known on other rivers. The data on which to base the value of C for the Miami are not very satisfactory. In the papers of the U. S. Geological Survey there are 4 years, 1906 to 1909, inclusive, for which floods at Dayton are recorded. An average of these four floods would indicate a value of C of about 86. The best indication of the average flood at Dayton is that deduced from the gauge heights of the U. S. Weather Bureau, which are available for 21 years. As the rating curve for large floods is indefinite, it seems best to obtain the average flood by means of the median gauge height during this period. This has been 11.9, corresponding to a flood of about 39 000 cu. ft. per sec., according to the rating by the U. S. Geological Survey in 1906. During the last 10 years the median gauge height has been 13.25, which corresponds to a flood of about 45 000 cu. ft. per sec. From other rivers it has been found that the median flood is usually less than the average by about 10 per cent. On this basis the average yearly flood on the Miami at Dayton, on the 24-hour basis, under present conditions, is probably at least 45 000, and the value of C is at least 90. The corresponding maximum rate of flow would be 50 000 cu. ft. per sec., or more.

During the past 20 years many changes have been made in the channel of the Miami, such as encroachment on the river channel in cities, and the construction of bridges with long approaches of solid embankment, which greatly reduce the waterway. It seems probable that the coefficient of C for the Miami River at the present time is different from what it was a number of years ago. Mr. Morgan thinks that the Miami should have a comparatively low coefficient of flow on account of the comparatively small average slope of the water-shed. Steep slopes on the upper branches with smaller slopes on the main

Mr. Fuller.

river and lower branches are conditions which may produce larger floods than uniform steep slopes. A comparison of the coefficients for different rivers of the same general character fails to indicate any great effect from the average slope of a river. The writer suggests that the relation between the slopes on different parts of the river and on its branches is of greater importance in determining the flood-producing capacity than the average slope.

The maximum flood on the Miami in 1913 may have been very different in magnitude from the flood which would have occurred if the channel had been unobstructed. The large number of embankments thrown across the valley, with inadequate waterways, provided temporary storage which tended to retard the earlier run-off until the branch streams poured in their maximum flood flows. The subsequent failure of these embankments releasing the stored waters, probably increased the flood materially.

Taking these matters into consideration, together with the great uncertainty of gauging such a flood, it seems probable that the recorded flood flow may have been considerably in excess of the flood which would have come under natural conditions. On the whole, the writer believes that the Miami flood of 1913 was not greater than five times the average yearly flood under present conditions, and that it may have been much less. At all events, the flood was a very exceptional one, and must be given careful consideration in determining the probable floods on our rivers, particularly those in the central part of the country.

The flood on the Hudson River in 1913 is an interesting one, from the standpoint of what may happen on other rivers. The maximum flood on the Hudson, for which records for many years are available, was, previous to this year, much less than what would normally be expected on the basis of its average flood. The flood of 1913, however, not only reached the normal maximum flood for the period of record, but exceeded it by a considerable quantity. There are many other rivers on which the maximum recorded flood is much less than the flood indicated by the average relation. That much greater floods will occur on some of these rivers within a comparatively short period seems assured. As an instance, the greatest flood on record on the Ohio at Wheeling and at Pittsburgh is equivalent to a flood which may normally be expected in a period of not more than 15 years. If a flood of the relative size of that on the Hudson occurs on the Ohio, it will mean a flood of more than 600 000 cu. ft. per sec. at Pittsburgh, or a flood from 4 to 5 ft. higher than the highest now on record.

Reliability of Frequency Relation as Established from Records on One Stream.—Messrs. Knowles, Horton, and Pillsbury give frequency relations for individual streams, which differ from the average relation

proposed by the writer. To show how unreliable such relations may be, Figs. 17 and 18 are presented, which show how greatly one or two of the larger floods affect the relation, even when derived from the longest records available for any stream. Mr. Fuller.

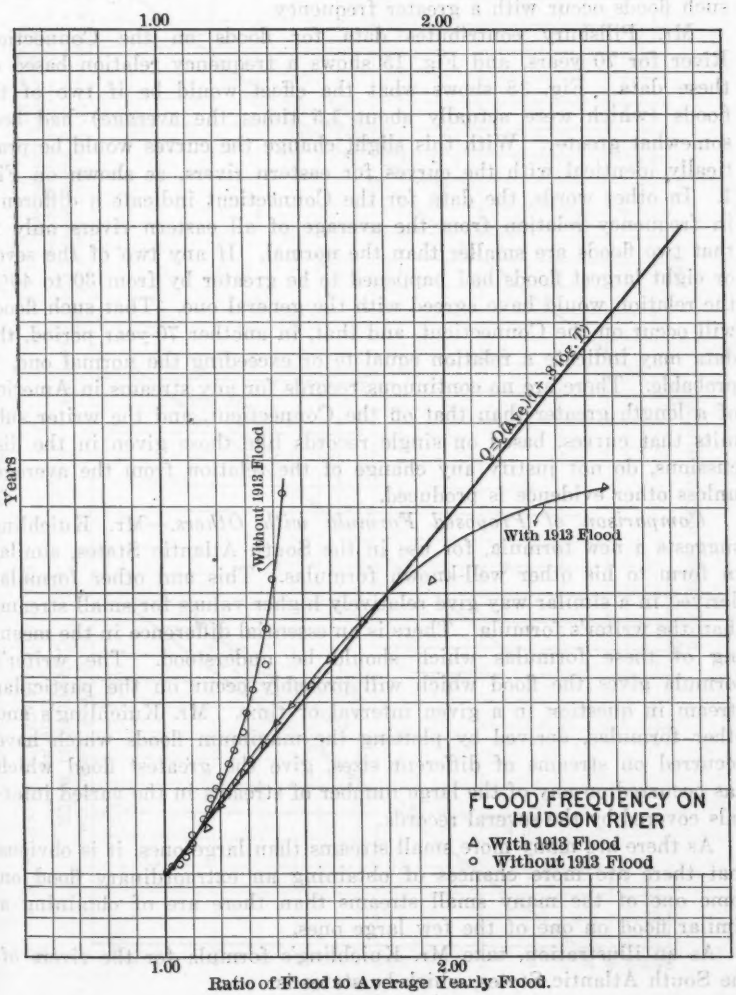

FIG. 17.

On Fig. 17 two curves illustrate the effect of a single great flood. The flood of 1913 on the Hudson is the greatest of which we have record. The effect of this flood is sufficient to change the data so that

Mr. Fuller.

the indications are just the reverse of what they were before the flood occurred. In other words, prior to 1913, the Hudson River records indicated that great floods occurred less frequently than on the average river. By including this flood in the data, the indications are that such floods occur with a greater frequency.

Mr. Pillsbury contributes data for floods on the Connecticut River for 70 years, and Fig. 15 shows a frequency relation based on these data. Fig. 18 shows what the effect would be if two of the floods (which were actually about 1.5 times the average) had been somewhat greater. With this slight change the curves would be practically identical with the curves for eastern rivers, as shown on Fig. 1. In other words, the data for the Connecticut indicate a difference in frequency relation from the average of all eastern rivers only in that two floods are smaller than the normal. If any two of the seven or eight largest floods had happened to be greater by from 30 to 40%, the relation would have agreed with the general one. That such floods will occur on the Connecticut, and that, in another 70-year period, the data may indicate a relation equal to or exceeding the normal one, is probable. There are no continuous records for any streams in America of a length greater than that on the Connecticut, and the writer submits that curves, based on single records like those given in the discussions, do not justify any change of the relation from the average, unless other evidence is produced.

Comparison of Proposed Formula with Others.—Mr. Kuichling suggests a new formula, for use in the South Atlantic States, similar in form to his other well-known formulas. This and other formulas derived in a similar way give relatively higher values for small streams than the writer's formula. There is an essential difference in the meaning of these formulas which should be understood. The writer's formula gives the flood which will probably occur on the particular stream in question in a given interval of time. Mr. Kuichling's and other formulas, derived by plotting the maximum floods which have occurred on streams of different sizes, give the greatest flood which has occurred on any of the large number of streams in the varied intervals covered by the several records.

As there are many more small streams than large ones, it is obvious that there are more chances of obtaining an extraordinary flood on some one of the many small streams than there are of obtaining a similar flood on one of the few large ones.

As an illustration, take Mr. Kuichling's formula for the rivers of the South Atlantic States, which he states is

"based on the greatest observed discharges of the Potomac River at Point of Rocks, Md., the New River at Radford, Va., the Catawba River at Rock Hill, N. C., the Little Tennessee River at Judson, N. C., Cane Creek at Bakersville, N. C., and numerous other streams which

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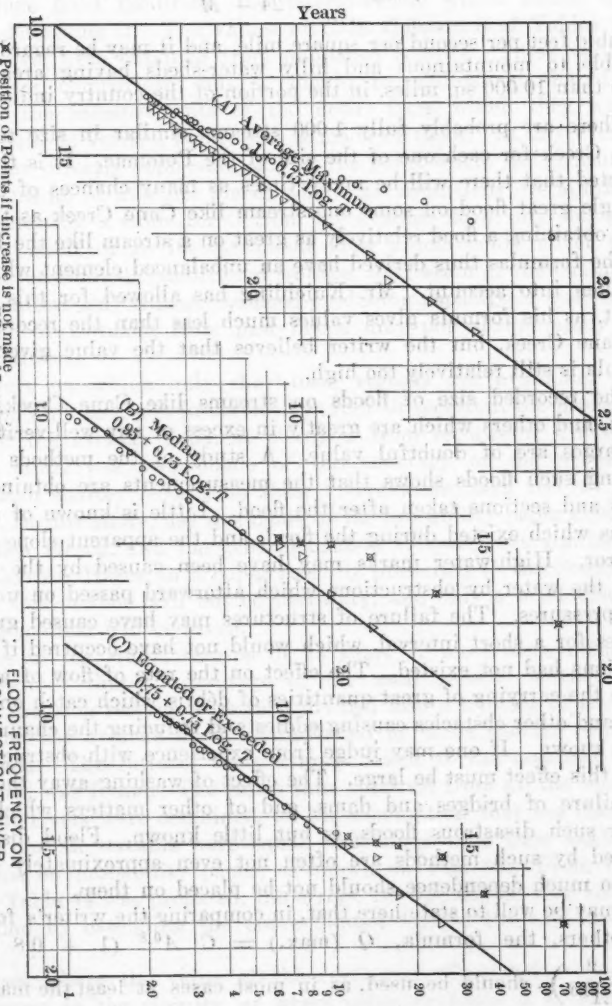

Fig. 18.

Mr. Fuller. exhibit somewhat smaller rates of discharge than the preceding. This new formula is

$$q_{max.} = \frac{41.6\ (620 + M)}{24 + M} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in cubic feet per second per square mile, and it may be regarded as applicable to mountainous and hilly water-sheds having areas of not more than 10 000 sq. miles, in the portion of the country indicated."

There are probably fully 1 000 streams similar in size to that of Cane Creek for each one of the size of the Potomac. It is then to be expected that there will be many times as many chances of obtaining a single great flood on some one stream like Cane Creek as there will be of obtaining a flood relatively as great on a stream like the Potomac.

The formulas thus derived have an unbalanced element which must be taken into account. Mr. Kuichling has allowed for this to some extent, as his formula gives values much less than the recorded flood on Cane Creek, but the writer believes that the value given by the formula is still relatively too high.

The recorded size of floods on streams like Cane Creek, Devil's Creek, and others which are greatly in excess of any well-verified flood discharges are of doubtful value. A study of the methods used in gauging such floods shows that the measurements are obtained from slopes and sections taken after the flood. Little is known of the conditions which existed during the flood, and the apparent slope is often in error. High-water marks may have been caused by the backing up of the water by obstructions which afterward passed on under the flood pressures. The failure of structures may have caused great discharges for a short interval, which would not have occurred if the obstructions had not existed. The effect on the rate of flow of a stream due to the carrying of great quantities of débris which catch on fences, trees, and other obstacles causing eddies and reducing the channel area, is not known. If one may judge from experience with obstructions in pipes, this effect must be large. The effect of washing away the banks, the failure of bridges and dams, and of other matters which occur during such disastrous floods, is but little known. Flood discharges obtained by such methods are often not even approximately correct, and too much dependence should not be placed on them.

It may be well to state here that, in comparing the writer's formulas with others, the formula, Q (max.) $= C\ A^{0.8}\ (1 + 0.8 \log. T) \left(1 + \frac{2}{A^{0.3}}\right)$, should be used, as in most cases at least the maximum rate of flow is given by the other formulas.

Selecting a Value for C.—The selection of the proper value of C is important in the use of the proposed formula. Before its selection, a study of the flood data on the stream in question and on other

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streams of similar nature in the vicinity should be made. This study should be just as complete and thorough as it would be if any other method of determining the maximum flood were to be used. The use of the average flood facilitates the determination where means are at hand to determine it. The values of C in Column 8 of Tables 12 to 26, inclusive, were obtained from published records. These values are useful for comparison, but it should be understood that the accuracy of the measurements of the floods from which they were obtained should be verified before use is made of them. In any event, coefficients are directly applicable only at the point where the measurements were made, or at other places where the conditions are similar.

Before selecting a value for C from the average flood, any changes in the river itself or in its catchment area which would affect the value should be studied. Additional storage, congestion of the channel, and other matters may change the flood-producing capacity of a stream, and a value of C taken from past floods may not always apply to present conditions.

For rivers on which only short-term records are available, the average flood may be much in error. In order to ascertain the probable accuracy of the average flood, as obtained from records of different lengths, a study has been made of the accuracy of the averages obtained from shorter records on those rivers where long records are available. In this study, use was made of the probability paper devised by Mr. Allen Hazen, and the general method followed was similar to that used by Mr. Hazen in a paper recently presented to the Society.* Details of this method will not be necessary, and only a brief discussion of the method will be given.

For use in the study, records of 15 years or more were utilized. It was assumed that the average of the total records is the true average. The long records were then divided into a number of shorter records, and the average flood, as indicated by these shorter records, was obtained. The ratio of these average floods to the assumed true average flood was found. The ratios thus obtained from all the records of different rivers were combined, on the assumption that the chances for errors on all streams were equal. Plottings were then made of these ratios on probability paper, as shown on Fig. 19. From these plottings Table 44 was prepared, the errors, for 20- and 25-year records being found by extending the curve plotted for 5-, 10-, and 15-year records.

In cases where the value of the coefficient, C, is below that of other streams in the vicinity, or where other evidence is at hand to indicate that the coefficient, C, is too low, an increase in the value of C by at least the probable error would be justified. A further increase

* "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply," *Proceedings*, Am. Soc. C. E., November, 1913.

Mr. Fuller. to provide a factor of safety depends largely on the importance of the works under consideration, and on how great a factor of safety has otherwise been provided for in the selection of the value of T.

TABLE 44.

Length of the record, in years.	Probable error. The chances are 1 in 4 that the true average will exceed the average obtained by the record by the percentage given below.	The chances are 1 in 10 that the true average will exceed the average obtained from the record by the percentage given below.
5	11½	22
10	7.5	14
15	5.5	10
20	4.0	7½
25	3.5	6½

Selection of the Value of T.—In the paper the writer has given his views on the method of selecting the value of T. He wishes, however, to call attention to the necessity of using a large value for all important work. The use of a large value of T is much the same as the use of a factor of safety in other engineering works. If we should build structures on a thousand different rivers, using a value of T equal to 1 000, we should do so on the expectation that in each 10 years some five of these structures would be called on to stand a flood in excess of that provided for. For large and important structures, the failure of which would be disastrous, a larger value of T than 1 000 should be used. Where the cost is not prohibitive, providing for floods of from four to five times the average yearly flood does not seem unreasonable for such works.

Mr. Morgan suggests that during some years storms occur which cause great floods on many different rivers, and that this may influence the frequency relation. Undoubtedly more great floods do occur in some years than in others. If the study were based on floods on streams of similar size in one section of the country, the effect of these periodic storms might be great. The writer does not believe that the effect of such periodic storms has materially influenced the proposed general frequency relation, for the following reasons: In deriving the relation, streams of greatly different size are utilized. Great floods on small streams are due to very heavy local storms which do not necessarily affect the largest rivers. The streams considered are widely distributed. During each year, on some streams, floods occur greater than any previously recorded. Some of the records used in establishing the relation go back from 50 to 100 years, and the records are pretty well distributed over the last 20 years. The ten greatest relative floods, as given in Table 9, occurred in eight different years. A study of Columns 7 and 8 of Tables 12 to 26, inclusive, shows no difference in frequency between streams with long and with short records.

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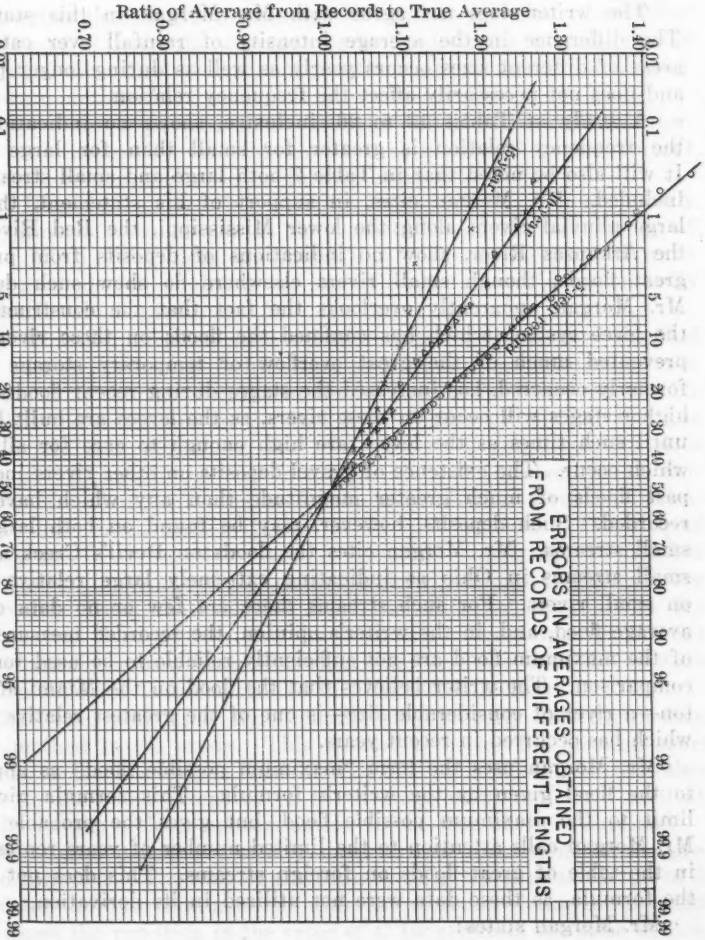

FIG. 19.

Mr. Fuller.

Mr. Morgan states:

"On large water-sheds of 10 000 sq. miles or more, excessive rainfall in one part of the water-shed is usually balanced by lack of rainfall in another part, and the ratio between the average annual flood and the maximum possible flood must be less than for small areas."

The writer does not agree with Mr. Morgan in this statement. The difference in the average intensity of rainfall over catchment areas of different sizes occurs yearly as well as during longer periods, and does not necessarily affect the frequency relation.

A study of Tables 12 to 26, inclusive, shows no indication that the frequency relation is greater for small than for large rivers. It will also be noted that in Table 9 both large and small streams are included. Mr. Morgan cites, in support of his statement, that the large alluvial rivers along the lower Mississippi, the Red River and the Arkansas River, show no indications of deposits from previous great floods, though small rivers elsewhere do show such deposits. Mr. Morgan apparently overlooks the fact that the construction of the levee system, which has confined the floods on these rivers, and prevented much of the great overflow or temporary storage which formerly occurred, has increased the stages during recent floods. Still higher stages will occur on these rivers, as the levees are built higher, until such times as the levees are high enough to care for all floods which occur. The existence of gravel deposits on other rivers indicates past floods of much greater magnitude than any which have been recorded. Such deposits, however, may be found on both large and small streams. Mr. Morgan cites the floods on Devil's Creek and on small streams in Ohio as indicating extremely large relative floods on small rivers. For such streams there are few or no data on the average flood, and, in the writer's opinion, the recorded measurements of the maximum flood are not sufficiently reliable to be used for such comparison. The writer believes that the flood on the Miami at Dayton—a river of considerable size—is one of the greatest relative floods which has occurred in recent years.

Mr. Morgan uses the term "maximum possible flood" as applying to the flood given by the writer's formula. This formula gives no limit to the maximum possible flood, but gives the probable flood. Mr. Morgan calls attention to the limited number of years represented in the table of great floods on foreign streams. This does not affect the formula, as these data were not utilized in its derivation.

Mr. Morgan states:

"It might be better to estimate the maximum possible flood by what might be called the rational method, that is, by determining from the basis of experience and the maximum rainfall to be expected, the relation of rainfall to run-off under the conditions which would exist in an assumed case, considering the elements of topography, shape of the drainage basin, direction of storms, season of the year, etc."

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There is quite as much uncertainty in probable maximum rainfall as in probable flood flow. There is, in addition, great uncertainty as to the percentage of run-off to be expected on different rivers, and under different conditions. We know but little of the effect of topography, direction of storm, shape of catchment area, and other factors. To the writer there seems to be much less chance for error in selecting the value of C from the average yearly flood, or by a study of the values of C for other rivers, than in estimating values for these many unknown quantities.

Many of the important points brought out in Mr. Morgan's interesting and instructive contribution have been covered in the writer's general discussion, and need no further comment.

Mr. Hinckley's interesting remarks, made in 1911, indicate clearly how engineers have looked at the probability of the occurrence of floods. His rainfall data are interesting. The application of probability methods for obtaining the probable rainfall would undoubtedly give much valuable information.

Mr. Chandler presents some valuable data for the flow of the Red River of the North and its branches, and draws some interesting conclusions as to the frequency relation of streams in that section. He says that the conditions in the Red River Valley are similar to those in a partly arid region, so that the writer's average relation does not hold. Although the data available are not sufficient to be conclusive, Mr. Chandler's suggested frequency relation gives an indication of how much higher it may be in such sections. The size of the average flood for such rivers is governed largely by the number of dry years in the period. It occurs to the writer that in determining the average flood, the exclusion from the record of all years so dry that no real flood occurs, may have merits, thus giving an average of the real floods as a basis of comparison. On this basis the relation may be expected to be more like that for other sections.

The writer notes that Mr. Chandler very properly uses the median flood method in his study. This method is better for plotting short records, as the few largest floods do not affect the plotting of the other floods to the same extent as in the average maximum flood method, and by its use a closer approximation to the true curve is obtained.

Mr. Hazen points out clearly the uses and limitations of the proposed formulas. The map of the country (Fig. 7) is most useful, as it indicates the data available for different sections of the country and also shows the variation in the value of C for rivers in the same section. A study of the conditions will in most cases account for such variations. For instance, in Maine, where the coefficients vary from 17 to 110, it will be found that the low coefficients are for small streams draining extensive systems of lakes which control largely the floods, and the large values are for streams on which there is little or no storage.

Mr. Fuller. It is interesting to study the effect of this storage on the river system. For the upper branches, the effect is great, but, as the streams join to form the larger rivers, the effect decreases rapidly until, near the mouth of the river, the coefficients are in some cases several times as great as for the upper branches. The study of streams in that State, where there are such extensive lake systems, is most illuminating in regard to storage for flood protection.

As Mr. Hazen states, probability paper might have been utilized in the study of flood frequency. During the study for this paper the normal probability curve was tried, but it was found that the data did not fit it as closely as the logarithmic curve adopted. The use of probability paper, which provides a ready means of drawing curves varying somewhat from the normal law, allows this method to be used. Practically, however, such a curve would give results identical with the formula, within the limits of the data, and an extension of the curve would indicate a ratio of 4.0 for $T = 10\,000$, as compared with 4.2 by the formula. Such differences are of little moment, and no data are available to indicate which is the more nearly correct.

The method of plotting gauge heights on probability paper, as suggested by Mr. Hazen, should prove useful. The writer has plotted records of the gauge heights for many stations along the Mississippi and its branches. Although these plottings are not in all cases as close to a straight line as the one for Cincinnati, they all approximate such a line. Within such limitations as may be applied by one having a thorough knowledge of the river, both as to changed conditions affecting the stages and as to the storage and increased channel capacity which occur at higher stages, this method seems applicable to many rivers.

Mr. Hazen's suggestion in regard to determining coefficients of variation, as an index of how closely the stream follows the average law of frequency, is interesting. It may be that a thorough study of the regularity of flow throughout the year, and of other characteristics of the stream, would enable the effect of some factors on flood frequency to be ascertained.

Mr. Knowles, citing a record of a single stream as an indication of variation from the average, warns against the general use of the average relation. The writer pointed out in the paper, and has further stated in this discussion, that a thorough study of all local conditions should be made before using the formula, but must repeat that, in his opinion, the data on a single stream are entirely inadequate for establishing a frequency relation. It could be shown by a plotting similar to Fig. 18 that the data for the Allegheny River at Kittanning really differ but slightly from the average relation. Mr. Knowles' extension of the existing record to a 41-year record, by estimating the flood at Kittanning from the gauge height at other points on the river

is of doubtful accuracy. An examination of Figs. 10 and 11 will show that the actual floods which Mr. Knowles plotted in order to obtain the curves showing the relation between the gauge heights at Kittanning and those at Freeport and Parker differ, in some instances, by from 50 to 100% from the curves. If these actual floods do not agree more closely with the curves, it is obvious that similar errors probably exist in the floods obtained in this way and included in the record. As the effect of the few large floods is so important, it is clear that a record obtained by such methods should not be used in discussing frequency. Mr. Fuller.

Mr. Bellamy's discussion is interesting, and gives much valuable information as to flood flows in Australia. The large variation in floods for the different rivers is to be expected in a country so large and with such widely varying conditions. As Mr. Bellamy states, a comparison of such rivers, with the limited data available, is very unsatisfactory. It is much the same as comparing rivers of the American coast, where the coefficients are in many cases greater than 100, with rivers in the Missouri River Basin or the Great Basin, where the coefficients are in many cases less than 10.

Mr. Kuichling has supplied tabulated data relating to great floods which have occurred on both foreign and American rivers, and these, taken in connection with those previously presented by him, give by far the most complete record of maximum flood flows available. The data from which this information was prepared are widely scattered. In many cases, particularly for foreign streams, the original data are in such a state that most careful study of them is required before they can be used. The collection and analysis of this large mass of data should be appreciated by all who are interested in the flood problem.

Mr. Kuichling brings out clearly many important aspects in regard to flood flows. Some of these points have been taken up by the writer in the preceding general discussion, and need no further comment.

Mr. Kuichling calls attention to the wide variation in the value of C for different rivers in the same general section of the country, and suggests that it may be better to use a single value of C for one section. Where coefficients are based on long records, the writer believes that the differences in the values of C indicate different flood-producing capacities for the streams. In many cases these differences may be accounted for by a study of the catchment area, as to slopes, storage capacity in lakes, and rainfall conditions. Where records of considerable length are not available, the writer agrees with Mr. Kuichling that much consideration should be given to the coefficients for other streams in the section. He is correct, in his interpretation of Tables 12 to 26, inclusive, in stating that the values of C in Column 8 were the ones intended for general use. The values in Column 8 are derived from the average flood, and take into account all the largest yearly floods in the rivers; they are more accurate than those in Col-

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umn 7. It would have been better, as he suggests, to have called these C_1 and C_2.

Mr. Kuichling calls attention to the important work of Iszkowski, and reduces his formula to conditions for the United States. Although this formula is of very different form from the writer's, it may be of interest to show how the results obtained from it agree with those obtained from the writer's formula,

$$Q\ (\text{max.}) = C\,A^{0.8}\,(1 + 0.8\ \log.\ T)\left(1 + \frac{2}{A^{0.3}}\right).$$

Under what may be regarded as similar conditions, Iszkowski's formula differs from the writer's by the following percentages: for a catchment area of 10 sq. miles, — 21%; 100 sq. miles, + 32%; 1 000 sq. miles, — 2%; 10 000 sq. miles, — 39 per cent. This comparison is made on the basis of the values deduced by Mr. Kuichling from Iszkowski's formula for "hilly country, slightly permeable soil, and sparse vegetation" and of a use of $C = 100$ and $T = 100$ in the writer's formula.

The writer has plotted the largest of the floods in America, as given by Mr. Kuichling, on the lower diagram of Plate LXIX, and finds that with the exception of three or four extreme floods, such as those on Cane Creek and Devil's Creek, the plottings agree with the curves representing the writer's formula as well as the points previously plotted.

Mr. Horton calls attention to a discussion of the use of probability methods in a report made by Mr. Rafter in 1896. This discussion is interesting and instructive. That paper, however, is entirely confined to a discussion of rainfall and minimum run-off. The writer fails to find any suggestion as to the use of similar methods for determining flood flows. Until Mr. Horton called his attention to it, the writer was not aware of the existence of this study. Mr. Horton further states that Mr. Rafter suggested to him the use of probability methods for determining the probable flood as early as 1896 and that he has used it on numerous occasions in his professional work. The writer had no means of knowing about any work which Mr. Horton had done on this subject, and sees no reason why he should qualify the statement that the original suggestion came to him from Mr. Hazen.

Mr. Horton gives three different formulas derived from the data on rivers near Philadelphia. He also gives the following general formula for which he states that he

"does not claim any great breadth of applicability for the general formula, * * * but believes that factors other than area modify the flood discharge of streams so profoundly that it is better, wherever possible, to derive individual formulas or flood-frequency diagrams for each stream."

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This formula is:

$$Q = 4021.5 \frac{T^{\frac{1}{2}}}{A}$$

in which Q is the flow in cubic feet per second per square mile. To put this formula on the basis of the writer's, that is, to give the total flood flow of the stream, it becomes necessary to multiply by A, so that the formula becomes

$$Q = 4021.5\ T^{\frac{1}{2}}$$

According to this formula, the maximum floods to be expected are independent of the catchment area. Surely a formula which does not take into account the size of the catchment area can have no general application.

During the early part of the study for this paper, exponential formulas such as those proposed by Mr. Horton were tried. It was found that though these gave curves closely following the data for short periods, for the longer records the exponential relation was less satisfactory than the logarithmic relation adopted.

The writer deems it unfortunate that, after seventeen or more years of consideration of the use of probability methods for the determination of flood flows, Mr. Horton should have confined his discussion so largely to criticizing details of the writer's methods, instead of giving more in regard to his own studies.

Mr. Horton states:

"It would appear that the relation between flood magnitude and frequency, when expressed in terms of either 'average maximum flood' or 'median flood,' is not only very indefinite and difficult of comprehension, but is apt to be very misleading, if it is not indeed practically meaningless. It does not directly convey the information which the engineer usually most desires, for example: If a spillway has a capacity of 1 000 cu. ft. per sec., how often on an average will its capacity be exceeded?"

In regard to this statement, the writer will call attention to the fact that the others who have discussed the paper have not found the proposed formula either indefinite or misleading, but, on the contrary, have comprehended its true meaning. The writer, on pages 1022-1029,* described three different methods which are applicable to flood frequency problems. In Table 6 the formulas representing these methods are stated, and values are given for the volume of the flood indicated by them in different periods of time. If any one wishes to know the volume of the flood that will probably be equalled or exceeded in a period of time, he may do so by using the formula:

$$Q \text{ (equalled or exceeded)} = C A^{0.8} (0.7 + 0.8 \log. T)$$

* *Proceedings*, Am. Soc. C. E., May, 1913.

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as given in Table 6. There are problems in which it is useful to know the volume of this flood, but the writer believes that the most important question which the engineer desires answered is: What is the probable flood for which to design our structures? For this purpose we surely do not wish to ascertain the smallest of the maximum floods that is likely to occur in the given interval, which is only a different way of stating the "flood to be equalled or exceeded." For example, suppose we have a record which includes ten periods, each of 20 years. Each of these periods would have a maximum flood of a different size. The flood to be equalled or exceeded in a 20-year period would be the smallest of these. To design works which would just provide for this flood would mean that in all probability the design would prove inadequate within the period, because during nine out of ten periods, greater floods would occur.

The average of the ten maximum floods which have occurred in the several periods, as given by the writer's formula, seems to be a logical basis for design in order to provide for a reasonable chance that the structure will survive the given interval. The median of these floods, or the one for which there is one greater for each one less, represents the flood for which the chances are even that it will occur in the given period. The use of this flood is as logical as the use of the average. The difference in value between the two is slight, and the writer, for the reasons stated in the paper, prefers the average maximum.

Mr. Horton, after giving a method of determining the "average interval of recurrence of a flood lying between any two given magnitudes", states: "This illustrates what seems to be a fatal error in the author's method of analysis."

The writer fails to find in the paper any statement which would justify the interpretation Mr. Horton has given the formula. As the method of derivation and the meaning of the formulas are explained in considerable detail in the paper, the writer considers that to apply it as Mr. Horton has done is entirely unwarranted. If the determination of the average interval of recurrence of floods of a size between fixed limits is desirable, as it may be for some special problem, the use of the flood to be equalled or exceeded, as suggested by Mr. Horton, is proper. Such use, however, is an unusual one, and to find a "fatal error" in the average maximum flood formula because it is not applicable to such a problem, is like condemning a formula proposed for determining the strength of a steel beam because it does not give the deflection of the beam. The writer was aware that the "flood to be equalled or exceeded" and the "median flood" had their uses, and accordingly gave the formula by which they might be obtained and descriptions of the methods of derivation, but, to

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explain their application to special problems, he thinks is beyond the scope of the paper.

Mr. Horton considers the use of ratios "confusing and cumbersome". By the use of ratios it becomes possible to compare the frequency of occurrence of floods on different rivers, which, in the writer's opinion, is of the greatest importance.

Mr. Horton objects to the writer's method in that he does not use all the floods to determine the frequency relation. As a matter of fact, in the determination of the general formula, 1 672 floods were considered; 20 of the largest were plotted individually and 200 others in groups of ten. This plotting covered three-quarters of the total length of the curve, as plotted in Fig. 1. It is apparent that, with the average maximum method, the average flood, which has a ratio of unity, must be plotted for 1 year. A straight line drawn from unity through the average of the points plotted satisfied the requirements, thus making the plotting of the remaining points so obviously unnecessary that it was not done. The writer assures Mr. Horton that the remaining points will fall close to the line. Mr. Horton's other criticisms of the paper are, the writer believes, fully covered by the foregoing discussion.

Mr. Pillsbury's discussion is an interesting and natural one. An examination of Fig. 18 shows how a slight change in the data on the Connecticut River would affect the frequency relation, and the writer has already given his views on the reliability of frequency relations established from the records of single streams. Aside from this, the assumption made by Mr. Pillsbury contains a fundamental error which would have to be, and could be, eliminated. This error is in assuming that the series of annual floods follows the normal law of probability. By the normal law of probability, the variations upward and downward from the normal are equal. As a matter of fact, with flood data, the variations upward and downward are not equal. The variations downward are more numerous, but the variations upward are greater in magnitude. The values follow, not the curve of normal probabilities, but what is called a skew curve. If Mr. Pillsbury will deduce the formula that most accurately represents this skew curve, and will show its application, he will give a useful solution of the problem which he has attempted, and will be doing the Profession a real service.

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CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN.

Discussion.*

By Messrs. W. D. Maxwell and Walter M. Smith, Sr.†

Mr. Maxwell.

W. D. Maxwell, Assoc. M. Am. Soc. C. E. (by letter).—The question of temperature changes in arches has been mentioned by several who have discussed this paper.

When the Walnut Street Concrete Bridge, of which the writer was one of the designers, was built in Des Moines, Iowa, in 1911, nine electrical resistance thermometers were embedded at various points in one of the arches. This arch has a clear span of 68 ft., a crown thickness of 16 in., and a solid spandrel fill. The greatest range in temperature was at the crown, where an average of two thermometers showed a temperature of 86° Fahr. about 16 hours after pouring, 9° below zero as the low point during the following winter, and 86° in the summer of 1912, one year after the bridge was built. Other thermometers placed in thicker portions of the arch showed somewhat less susceptibility to atmospheric changes. The average of two instruments embedded about 2 ft. in heavy concrete near the pier and abutment showed 98° Fahr. about 36 hours after pouring, 20° the next winter, and 85° the following summer. The average of all the thermometers for the first 6 months, gave a range of temperature in the concrete of 87.6 degrees. The upper limit of this range was the extreme produced by the heating of the setting concrete. The average of all the thermometers for the second 6 months was 79.2 degrees.

The extremes of atmospheric temperature in the summer of 1911 and the following winter were very great for this locality, the total

*Continued from February, 1914, *Proceedings*.

†Author's closure.

Mr. Maxwell.

range being 135 degrees. However, ranges of considerably more than 100° may be expected every year.

Elevations were taken on all six arches of the bridge in September, 1911, when the average of all the thermometers was 75°, and again in January, 1912, when the average was 11 degrees. The difference corresponded closely to that computed for a change of 64° in the temperature of the concrete.

In the design of the bridge, allowance was made for a 40° rise, and a 40° fall. These amounts were considered excessive by some engineers, but the actual observations show that they were not only not too small, but that the fall was more than 40°, the concrete having been poured at a general average of about 70 degrees.

Considering these temperature ranges, it is apparent that concrete bridges built in these rigorous climates, especially the hingeless arches, must be very carefully reinforced against temperature stresses, and it is also evident that the warmer climates and those having nearly uniform yearly temperatures possess a real advantage with this type of construction.

Mr. Smith.

WALTER M. SMITH, SR., M. AM. SOC. C. E. (by letter).—Mr. Gregory states that the decision to adopt a three-hinged arch for the Traver Hollow Bridge was taken because the foundations for the abutments and piers would be on compressible material. He also states that the preliminary estimates indicated that the three-hinged arch would be somewhat more expensive than the fixed-arch type. He is correct in the first statement, but is mistaken in the second. Only a rough estimate was made for a fixed arch at this site, making certain assumptions as to the depth to rock. Careful designs and estimates were then made for a three-hinged arch, to compare with the other. The writer had charge of both estimates, and made the design and estimate for the three-hinged arch. He has copies of these estimates before him as he writes. If the same unit prices are taken for the three-hinged arch as for the fixed arch, the saving would be about 30% for the former. Therefore the average increase in unit price for the three-hinged arch would have to be 43% in order that it might cost as much as the fixed arch for this site. When these estimates were obtained, as there was no information at hand as to the depth to rock, it was decided to build the three-hinged arch, as it was certain that the material would carry the load of an arch of this type by compressing slightly, and would not injure the arch.

Figs. 10, 11, and 12 show the details of the hinges and the method of connecting them to the steel ribs of the arch. It will be seen that the steel forms a continuous structure between the abutment hinges, with an additional hinge at the crown.

When the design of the Rye Outlet Bridge was under consideration, the writer also investigated a three-hinged arch for this site, at Mr.

Mr. Smith.

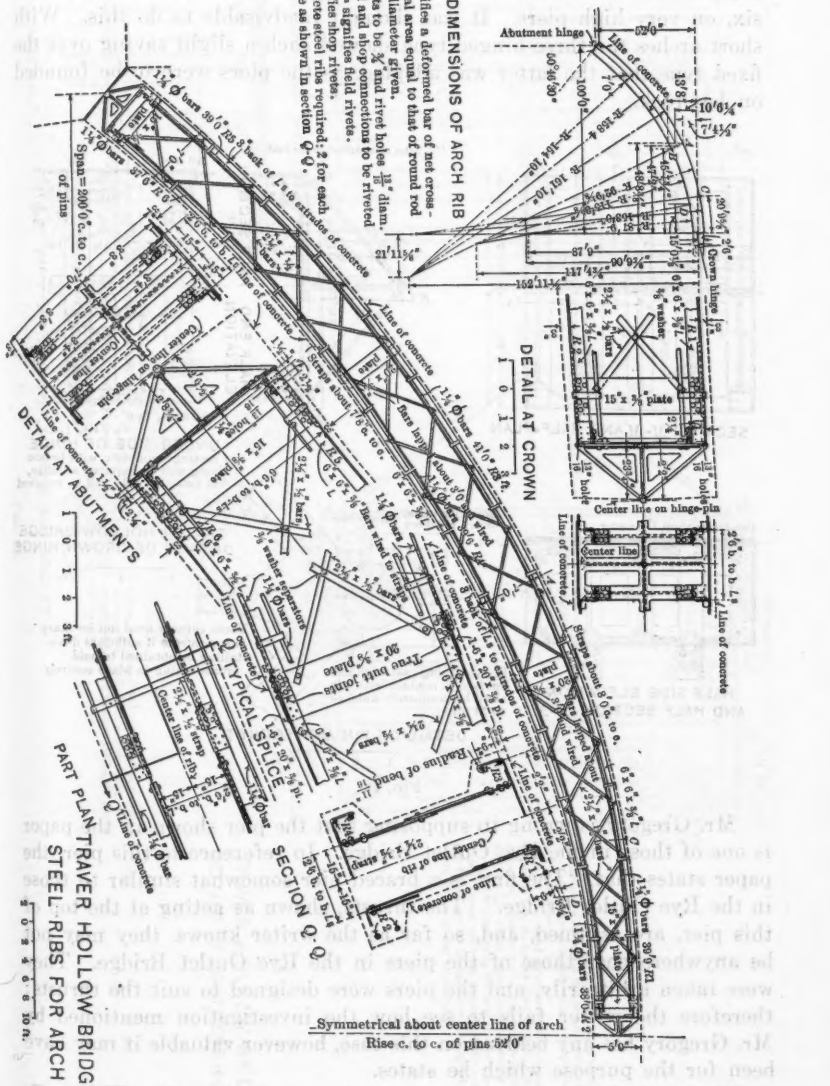

FIG. 10.

Mr. Smith.

Gregory's request. As the height of the roadway was fixed, and also the water line, the design was limited to short arches, unless it was admissible to construct them with a ratio of rise to span of more than six, on very high piers. It was thought inadvisable to do this. With short arches the three-hinged type showed such a slight saving over the fixed type that the latter was adopted, as the piers were to be founded on bed-rock.

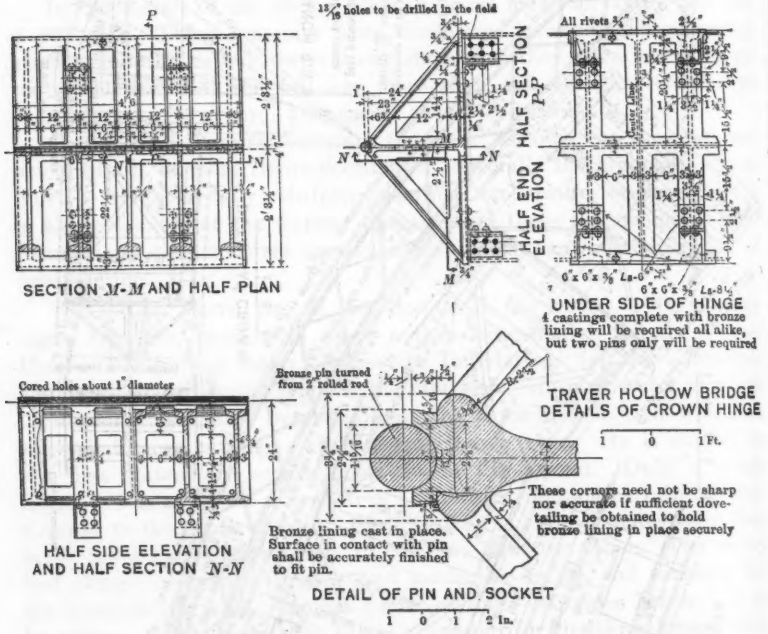

FIG. 11.

Mr. Gregory is wrong in supposing that the pier shown in the paper is one of those in the Rye Outlet Bridge. In reference to this pier, the paper states that, "The first is a braced pier somewhat similar to those in the Rye Outlet Bridge." The thrusts, shown as acting at the top of this pier, are assumed, and, so far as the writer knows, they may not be anywhere near those of the piers in the Rye Outlet Bridge. They were taken arbitrarily, and the piers were designed to suit the thrusts; therefore the writer fails to see how the investigation mentioned by Mr. Gregory has any bearing on this case, however valuable it may have been for the purpose which he states.

The method of constructing an arch, described by Mr. Wilson F. Smith, is a very satisfactory one, if the arch is to be of plain concrete,

Mr. Smith.

TRAVER HOLLOW BRIDGE
DETAILS OF ABUTMENT HINGE

FIG. 12.

Mr. Smith.

but, with all due respect to Mr. Janni, who states that he has constructed reinforced concrete arches in this manner, the writer cannot believe that it is economical. The difficulty of fitting the great number of bulkheads around the steelwork is very great, and the expense very heavy. In Pittsburgh, Pa., a reinforced concrete bridge, with a main arch of 216 ft., consisting of two separate ribs, is now being built by this method. With slight modifications, the same method has also been used extensively in America, and also in Germany, for plain concrete arches, and has proved very satisfactory.

The main arches of the Walnut Lane Bridge, in Philadelphia, Pa.,* were built in sections, leaving spaces between many of them to be filled with concrete later, as keys. Mr. Janni states that he has used the same method.

The arches of the Connecticut Avenue Bridge, Washington, D. C., were also built in sections, the spaces between the sections being filled in afterward, but this bridge and also that at Walnut Lane were of plain concrete, and in neither case was a temporary hinge made at the abutment or crown, as in Mr. Morison's method.

On the other hand, many masonry bridges in Germany have been built with temporary stone hinges at the crown and the abutments to insure the passage of the equilibrium polygon through these points for the dead load, the spaces around these hinges being filled with concrete on the completion of the bridge.

Mr. Quimby and Mr. Mensch state that, in the fixed arch and the three-hinged arch, the thrust at the crown is about the same, and therefore Mr. Quimby reasons that the thickness of the crown should be about the same, seeming to forget the fact that, for a heavily unbalanced live load, the thrust at the crown does not pass through the center, though in the three-hinged arch it is compelled to do so. In the Rye Outlet Bridge, which was designed for very severe conditions—a 20-ton road roller concentrated on one rib—this was especially so. At the crown the thrust was about 5 in. from the center; at the abutments, it was more than 12 in.; thus it was necessary to have a thickness of at least 30 in. at the crown and more than 6 ft. at the abutments in order to keep the equilibrium polygon in the middle third. As a matter of fact the thrust at the crown is slightly less in the three-hinged arch, on account of the difference in weight of the arch itself, but the great difference is caused by the thrust being central in the latter case. In the Rye Outlet Bridge, if three-hinged, only a thickness of 20 in. is needed at the crown and 25 in. at the abutments, as the thrust passes through the center at both places, and that at the abutments is only 25% greater than that at the crown.

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* "Walnut Lane Bridge, Philadelphia," by George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXV, p. 423.

Mr. Smith.

The point made by Mr. Quimby, that thickening the arch to keep the equilibrium polygon within the middle third will increase the temperature stresses on account of the added stiffness, is well taken, and this is one of the main objections to the fixed arch. How is this trouble to be remedied? Some engineers do so by assuming much less temperature change than the writer believes to be justifiable, thus letting the concrete crack and cause some kind of a hinge of its own if its strength is exceeded. Others do so by allowing higher unit stresses in the steel and concrete. Apparently, Mr. Quimby prefers the former method, judging from his statement of what he believes to be the maximum temperature variation in concrete.

The cracking of the concrete at the abutment or crown, or both, which occurs frequently in fixed-arch bridges for which a sufficient variation of temperature has not been allowed, will not necessarily endanger the arch. It is merely an attempt, on the part of the concrete, to remedy the fault of the designer, and form hinges of its own, thereby readjusting the stress. These cracks, however, are unsightly, and, when they occur, it is hard to convince the layman that the bridge is still safe.

The writer has stated that, with regard to appearance, the three-hinged arch is at a disadvantage, and, when the question of beauty is of sufficient importance, it would probably not be considered, its advantage being where economy is of first importance.

The maximum range of temperature change in an arch is not necessarily the change from midsummer to midwinter, with the condition of "no stress" lying midway between. Unless the arch is built in the spring or fall, and some method (such as that described by Mr. Janni) is adopted to eliminate the stresses due to setting shrinkage by leaving in the arch key spaces to be filled in after the concrete of the arch has been placed, set, and cooled off, this range will be much higher. If these precautions are not taken, the condition of "no stress" in the arch is the position it occupies at the setting temperature of the concrete, and any variation from this position causes deformation of the arch, and, therefore, stress. The variation will be all one way if the bridge be built in the summer. The steel in the arch has no effect in preventing cracks in the concrete, in fact, it generally causes them to occur more quickly than if it were not used. Many engineers have had this experience in building various structures. In building the retaining walls for the Pennsylvania Railroad terminal, in New York City, it was found to be impossible to make the length of the sections of wall greater than 25 ft. without cracks.* The reinforced concrete floor beams, which carry the concourse floor of vault light construction in the Pennsylvania Railroad Station, New York City, show cracks where they join the columns at almost 50% of the points. These were built as continuous beams over great distances, and the setting shrink-

* *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 334.

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age caused the cracks. The beams are not necessarily unsafe because of these cracks.

In 1908 a high, heavily reinforced concrete retaining wall was built along the railroad yard on the water-front at St. George, Staten Island. It is several thousand feet long, and was built without expansion joints. In the fall of that year the writer was informed, by one of the most prominent engineers connected with this work, that expansion joints had not been provided as they were not needed, the quantity of steel being sufficient to prevent it from cracking. The writer had seen some of the effects of this before, so in March, 1909, he inspected carefully about 1 000 ft. of this wall, and in that length counted 45 cracks extending from the base to the top, and 17 extending part way, making an average of about one crack in every 16 ft. Through 8 of these cracks water was issuing at various heights.

A similar experience was had in the top of the Cross River Dam. George G. Honness, M. Am. Soc. C. E., Division Engineer in charge of its construction, states that:*

"It was not expected that the use of these rods would entirely eliminate the temperature cracks, but it was hoped that it would prevent their concentration in a few large cracks.

* * * * * * *

"The reinforcement * * *—as to quantity and method of placing—was not effective in preventing the concentration of the cracks."

The following is given as an example of how greatly concrete shrinks in setting. On some work with which the writer was connected, some years ago, it became necessary to cut out some concrete in a ceiling consisting of a very deep and heavy concrete slab with reinforcing bars embedded about 2 or 3 in. from the lower surface. The concrete was cut from around a few of these bars for a distance of about 2 ft. All the bars, when released, sprang out of line, thus showing that the shrinkage of the concrete, in that short distance, was sufficient to shorten the steel bars by compressing them. Theoretically, these bars were carrying a tension of 16 000 lb. per sq. in. In reality, they were in compression, and the concrete was not only carrying all the load, but was also compressing the steel.

Many experiments have been made in order to determine the rise in temperature in cement mortar, neat and of various compositions, and of concrete during setting. In some tests by the Universal Portland Cement Company on neat cement cubes, the average temperature rise was from 80° to a little more than 130° Fahr., in from 6 to 8 hours after placing. In the case of concrete, the initial temperature, 70° Fahr., was increased to 80° in 8 hours, 100° in 25 hours, and 105° in 45 hours. Tests in the laboratory of Lehigh University have shown

* *Transactions*, Am. Soc. C. E., Vol. LXI, pp. 411-413.

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that the temperature of an 8-in. cube of 1:3 Portland cement mortar rose from an initial temperature of 70° to 160° Fahr. in 18 hours. At the Watertown Arsenal, a few years ago, tests of 29 cubes of neat cement, representing 27 brands of cement, were made. The initial temperature was 77°, and the average maximum 185° Fahr., showing a rise of 108° in an average of 13 hours. The thermophones embedded in the cyclopean masonry of the Boonton Dam indicated a rise of from 20 to 25° Fahr., although only about two-thirds of the mass consisted of concrete. In the locks of the Panama Canal the rise was also about the same.

Table 1 shows the setting temperature in some concrete canal walls at Madison, Me., built for the Hollingsworth and Whitney Company. As the concrete was placed in the winter, it was first heated to 70° Fahr. The average atmospheric temperature was 11° and that of the concrete 108°, showing a rise of 38° after being placed, in spite of the exceedingly low outside temperature.

TABLE 1.—SETTING TEMPERATURE IN CONCRETE CANAL WALLS AT MADISON, ME.

Date. 1909.	Depth of concrete, in inches.	Atmospheric temperature, in degrees, Fahrenheit.	Temperature of concrete, in degrees, Fahrenheit.
Jan. 19th	3.0	— 18	91
20th	3.0	— 2	94
21st	7.5	14	96
22d	15.5	34	98
23d	15.5	39	106
24th	15.5	20	110
25th	15.5	15	111
26th	15.5	18	113
27th	26.25	2	119
28th	35.0	18	119
29th	35.0	14	116
30th	37.0	21	116
31st	37.0	20	113
Feb. 1st	37.0	0	113
2d	37.0	— 5	110
3d	37.0	— 10	109
4th	37.0	— 11	113
5th	37.0	28	114
6th	37.0	28	106
7th	37.0	17	104
8th	37.0	6	101
9th	37.0	— 4	98
Averages		11	108

As an example of the low temperature which may be reached in bridge arches in the Northern States, Table 2, showing the results of tests on the Squaw Creek Bridge, at Ames, Iowa, is interesting.

Mr. Maxwell has given a brief account of the range of temperature experienced in the Walnut Street Bridge, Des Moines, Iowa, and

Mr. Smith.

readers are referred to a synopsis of his original report on this bridge, and also on some others.* Many of the data herewith presented have been obtained from Mr. Maxwell's valuable report. Tables 3 and 4, taken from that report, are interesting.

TABLE 2.—RESULTS OF TESTS ON SQUAW CREEK BRIDGE, AMES, IOWA.

Thermometer.	Maximum.	Minimum.	Range.
A1	82.2°	— 2.5°	84.7°
B1	79.5°	2.0°	77.5°
A2	80.5°	8.0°	72.5°
B2	81.4°	— 0.5°	81.9°
Average range			79.1°

TABLE 3.—TOTAL TEMPERATURE RANGE FOR THE WALNUT STREET BRIDGE.

Coil No.	Minimum temperature, in degrees, Fahrenheit.	Maximum temperature, in degrees, Fahrenheit.	Range of temperature, in degrees, Fahrenheit.
PERIOD FROM AUGUST, 1911, TO JANUARY, 1912.			
2	15	108	93
3	20	102	82
4	19	95	76
5	18	96	78
6	0	88	88
7	— 5	90	95
8	— 9	86	95
9	— 9	85	94
Average range			87.6
PERIOD FROM JANUARY TO AUGUST, 1912.			
2	15	84	69
3	20	83	63
4	19	86	67
5	18	85	67
6	0	87	87
7	— 5	86	91
8	— 9	84	93
9	— 9	88	97
Average range			79.3

The coils were in different portions of the structure, and at different distances from the surface. Coils Nos. 2 to 5 were placed in the concrete where the arch joins the abutments or piers. No. 1 in the

* *Engineering and Contracting*, November 12th, 1913.

TABLE 4.—RISE IN TEMPERATURE DUE TO SETTING. Mr. Smith.

Coil No.	Initial temperature of concrete, in degrees, Fahrenheit.	Maximum temperature of concrete, in degrees, Fahrenheit.	No. of hours after placing.	Rise of temperature, in degrees, Fahrenheit.
1.....	80	107	75	27
2.....	76	108	43.5	32
3.....	76	102	27.5	26
4.....	80	89	75	15
5.....	76	96	18	20
6.....	76	88	10	12
7.....	78	90	23	12
8.....	76	86	10	10
9.....	78	84	23	6
Averages.....	77.4	94.5	33.6	17.1

center of the pier, Nos. 6 and 7 in the center of the arch halfway between the springing line and the crown, and Nos. 8 and 9 in the center of the arch at the crown. The arch is not exposed on its upper side, as there is an earth fill above it. Coils Nos. 6 and 7 are about 9½ in. from the surface, and Nos. 8 and 9 about 8 in., consequently the heat generated by the setting would be dissipated much more rapidly at Nos. 6, 7, 8, and 9, than at the others. Conversely, these four reach temperatures much lower in winter and somewhat higher in summer than the others. As the upper surface of the arch is protected by an earth fill, the range there is probably not as great as it would be if the arch had open spandrels, therefore the variations are on the conservative side.

It is evident, in this case, that the concrete set and the arch took its shape at a temperature of more than 90 degrees. Any deviation from this temperature, then, would cause stress in the arch. As all the thermometers in the arch itself reached zero, or went below zero, during the following winter, the arch was subjected to a stress due to a variation of more than 90° from the normal. The arch was designed for a variation of 40° each way from the normal, therefore it was subjected to 2¼ times as great a range as that for which it was designed. What is likely to happen in such a case? Probably the arch will crack, either at or near the abutments, or at the crown, or at both. As the arch at the abutments or piers is much deeper, and therefore much more rigid, than at the crown, the cracks are more likely to appear there. When the concrete cracks the stress is readjusted, and the arch has formed somewhat of a hinge for itself. As stated previously, the arch is not necessarily endangered by these cracks, as they cause a readjustment of the stress, but they are unsightly, and it is difficult to convince a layman that the arch is perfectly safe.

In his report Mr. Maxwell states that at that time there were no

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cracks in the Walnut Street Bridge, but in the Squaw Creek Bridge there are unsightly cracks at the exact point of computed maximum stress at three of the four corners of the bridge. He also states that in the Locust Street Bridge, Des Moines, Iowa, there are cracks at the crown of each of the shore spans.

As all these arches were designed for a variation of 40° each way from the normal, Mr. Maxwell concludes that for northern latitudes a variation of this much should be allowed, and that:

"Particular circumstances may demand that a greater variation be used for drop in temperature to prevent the appearance of cracks."

This conclusion was reached by the writer some years ago. If, in the face of these facts, an engineer is willing to design a bridge for a variation of only 20° each way from normal, as Mr. Quimby seems to propose, and then lets the bridge crack and form its own hinges, the writer freely admits that it will be a cheaper bridge than a three-hinged arch designed for reasonable stresses.

It is admitted that, if a stone hinge be used in an arch, the friction of the hinges should be taken into consideration in computing the stress due to change of temperature, but with a metal hinge, such as that in the Traver Hollow Bridge, the amount would be so small as to be negligible. The writer would like to call Mr. Janni's attention to the details of these hinges and the arch rib, Figs. 10, 11, and 12. He will see that the span of the arch is 200 ft., and that the diameter of the pin at the abutment hinge is about 3 in., and at the crown 2 in.; therefore the maximum deviation of his lines, P_1 and P_4, would be 10 in. in 110 ft., which is certainly not very serious, to say the least. The pressure on these pins is about 12 000 lb. per sq. in. The socket for holding the pin is very heavy and rigid so as to cause very nearly an even distribution of pressure on the pin and a lining is cast in the socket and then finished to fit the pin accurately. The metal of the body of the hinge is cast steel having an elastic limit of 40 000 and a tensile strength of 76 000 lb. per sq. in., an elongation of 20% in 2 in., and a reduction of area of 41.5 per cent. A sample of this steel, ½ in. thick, was bent 180° around a diameter of 1½ in. without a sign of cracking either on the outside or inside of the bent portion. The socket of the casting was lined with "Monel" metal by casting it in place and then machining it to fit the pin accurately. The pin was of the same metal, finished from a rolled bar. This metal is a natural alloy of approximately the following composition: nickel 68%, copper 31%, and iron 1 per cent. The writer has a sample of this metal which tested as follows: tensile strength, 100 000 lb. and elastic limit 71 600 lb. per sq. in., elongation, 35% in 2 in.; reduction of area, 54.7 per cent. Now, with this metal as a bearing for the hinges, even if there was the concentration of pressure which Mr. Janni thinks is so serious,

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there would still be a factor of safety of 3 on the elastic limit of the material, and this, the writer thinks, is sufficient.

For the bending moment due to friction on the pin, the coefficient of friction of the pin and lining may be assumed at 0.20. The best authorities give this coefficient, for metal on metal, dry, as from 0.15 to 0.25; and, as this bearing is bronze on bronze, it will probably be nearer the lower limit, so that 0.20 should certainly be conservative. The total pressure on the pin is about 2 000 000 lb., therefore the bending moment of the friction at the surface of the pin, with the lever arm of 1½ in., will be $2\,000\,000 \times 0.20 \times 1.5 = 600\,000$ in-lb., which would be the moment caused by the pressure line deviating from the center line by less than 0.3 in., as the pressure decreases as the distance from the pin increases.

Mr. Janni speaks of taking care of the stress due to dead load and that due to shrinkage by neutralizing their effects, as much as possible, by the method mentioned previously, that is, by building the concrete of the arch in sections and leaving key spaces between them to be filled in after the cement has set—in not less than 10 days, he states. His reference to neutralizing the dead load stresses is supposed to mean the stresses caused by the pressure line for dead load leaving the center line of the arch and thus causing a bending moment, as direct compressive stresses caused by the dead load, of course, cannot be neutralized in any way. All that is necessary to accomplish this is to design the arch so that the center line is the pressure line for the dead loads on the arch. When the dead load has been obtained, it is very easy to do this. As stated previously, this is probably as good a method as can be found, for a fixed arch, to neutralize the stresses due to setting, but is, certainly, very expensive; and it does not affect the stresses due to a change from the atmospheric temperature at which the arch was cast to the extreme variation either way.

In any of the Northern States the average temperature for June, July, and August is higher than 70°, and the winter temperature goes well below zero at frequent intervals. Therefore, in any open spandrel arch built in the Northern States during these three months, the drop in temperature will probably be—as a conservative estimate—at least 60 or 65 degrees. Mr. Maxwell has shown that, in Iowa, at least, the range will be much more than that.

If all arches could be built between March 15th and May 15th, or between September 15th and November 15th, this trouble would not occur, and a method of neutralizing the stresses due to setting, similar to Mr. Janni's and allowing a range of 40° each way from the normal, would probably insure the arch against cracks.

In Mr. Aylett's Fig. 8, an arch of I-section, there would probably be some danger of the flanges breaking off, as he suggests. These flanges should be at least twice as thick as shown by Fig. 8, as it is

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not practicable to approach the thinness of the steel beam flange with concrete. The proportions used in the Traver Hollow Bridge are shown on Fig. 10.

The writer, however, cannot see that there is much danger of these flanges being broken off, as the load of the arch is carried to the falsework as a distributed load across its lower surface. The paneling of the sides is of light construction and is not independent of the falsework, therefore it has to move with it, and, as the movement of the falsework is caused by a corresponding movement of the arch, the whole structure must, of necessity, move together.

It may be of interest to state that the writer has a letter from the contractor who built the Traver Hollow Bridge, stating that no difficulty was experienced in its construction.

The writer will admit that, if piers are to be built in a flowing stream, in which there are quantities of tree trunks with the boughs all nicely trimmed off a short distance from the trunk, so that they can catch on anything with which they come in contact, as shown in Mr. Aylett's Fig. 9, there might be danger of a quantity of drift collecting around the piers. However, he does not believe that there is much danger of an object, after being deflected by the cut-water, being drawn in again quickly enough to be caught by the down-stream leg of the pier. He can see no danger whatever of a tree trunk being deflected by the cut-water and then running in again with its butt between the legs of the pier, and then swinging around and hanging there. The cut-water throws the water away from the pier, and, on account of this obstruction, the water will be slightly higher as it passes, and hence the velocity will be increased. Any object deflected by the cut-water will be carried away from the pier slightly by this elevation of the water, instead of being drawn in.

In streams subject to heavy floods, where any obstruction of the channel would be serious, it is probable that wide piers would be out of the question, and undesirable, no matter what the economy would be in theory. The writer does not wish to be understood as claiming that the braced pier shown should be adopted in all cases, by any means, for he believes that the conditions at every important bridge should be investigated carefully. He thinks, however, that the braced pier is worthy of investigation when the design for a bridge is taken up.

Mr. Mensch is mistaken if he thinks that the writer claims, as a general thing, that the concrete in a three-hinged arch is about 50% less than in one that is fixed, and no statement to that effect was made in the paper. This happened to be a case where the comparison was unusually favorable to the three-hinged arch, as the proportion of rise to span is almost 1 to 6. The writer used this arch because he had designed it some years ago for the Rye Outlet Bridge,

and had all the dimensions on hand, and wished to compare a three-hinged arch with it. There are five arches in the Rye Outlet Bridge, and this is the center span taken for a single-arch bridge. Mr. Smith.

It is an interesting fact that though this comparison is so favorable for the three-hinged arch for a single span, when it is applied to the Rye Outlet Bridge there is very little difference in cost, not more than about 4%, if one does not take into account the steel reinforcement. The reason for this is that the bridge piers are so high and the superstructure so massive that the total cost of the arch concrete is only 13% of the cost of the bridge, and $13 \times 0.48 = 6.25\%$ without allowing for the cost of the hinges. Taking these into consideration, the difference is reduced to about 4 per cent.

In stating the conclusions in the paper the writer did not mean to claim that every individual design would necessarily agree with these conclusions, but that they were true in the average case. He thinks that the paper has brought out a valuable discussion on a subject which, heretofore, has not been treated at length, and is glad that the matter was brought up. He has not found anything in the discussion, however, to cause him to change his conclusions, but has been further strengthened in them.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

By William Cain, M. Am. Soc. C. E.†

William Cain, M. Am. Soc. C. E. (by letter).—Mr. Janni makes a perplexing statement when he says: "cos. $(\alpha + \beta)$ is purely a symbol of a certain operation to be performed, and is not an actual value; its value is given by cos. α cos. β — sin. α sin. β". Perhaps this is a *"lapsus calami"*. All our textbooks state that the cosine of any angle, as $(\alpha + \beta)$, is an actual value and give tables of the actual values for various values of $(\alpha + \beta)$. Also, such values are often represented by certain lines on a unit circle, by aid of which the formula quoted is derived. By a consideration of such line values, it is at once seen that cos. $(\alpha + \beta)$ approaches indefinitely cos. β as α tends indefinitely toward zero, and the same result follows from using the development of cos. $(\alpha + \beta)$. Mr. Cain.

In fact, take the equation,

$$P\,Q = \frac{v\,\alpha}{\text{cos. } \beta - \alpha \text{ sin. } \beta},$$

which Mr. Janni gives. As he knows, α sin. β is an infinitesimal, and can be neglected in comparison with the finite quantity, cos. β, when connected with it with either a plus or minus sign. Thus, as α tends toward zero, without ever becoming zero, the value of $P\,Q$ approaches $\frac{v\,\alpha}{\text{cos. } \beta} = v\,\alpha \text{ sec. } \beta$, which is exactly the formula derived by the writer.

The writer cannot agree with Mr. Janni that the direction of the bending stress on the section, $N\,I$, varies in the manner shown in

* Continued from February, 1914, *Proceedings*.
† Author's closure.

Mr. Cain.

Fig. 9. It will presently be proved that the stress on the cross-section of an actual beam, varies continuously in amount and direction, so that no sudden change in direction of 90° at any point can occur.

This is true even at points of contact of wheel loads, for such loads are in reality distributed. So far as dams are concerned, the changes in the amount and direction of the principal stresses along horizontal sections have been determined, both theoretically and experimentally.

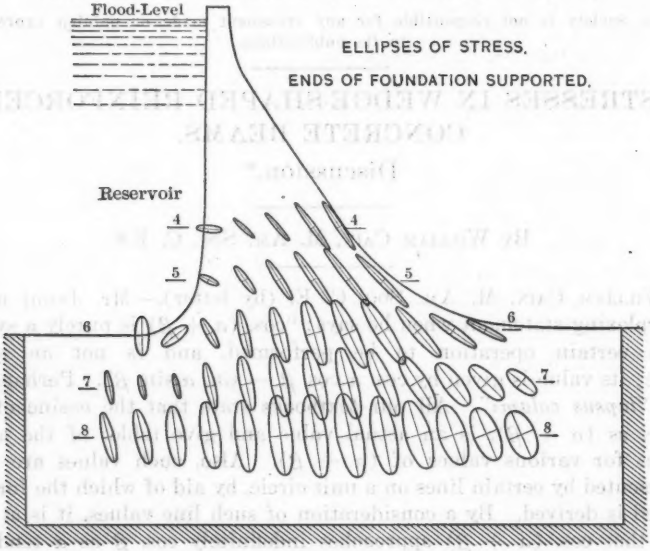

FIG. 13.

Reference has been made to the experiments on india rubber model dams by Messrs. Wilson and Gore. Fig. 13 is taken from their paper,* giving an account of these very interesting experiments. The unit stresses, in amount and direction, at numerous points in the dam, were determined from the distortions, and are represented by the ellipses of stress shown in the figure. The principal stresses, or those acting normally to their planes, are represented, in amount and direction, by the semi-axes of the ellipses. Of these, the maximum normal stresses are given by the semi-major axes and are those that specially interest us.

It will be observed that along any horizontal section, these unit principal stresses change gradually, in amount and direction. They are also observed to act parallel to the inclined down-stream face, very near that face. At the up-stream face, where the dam is subjected to

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907-08, Part II.

water pressure, the direction of the principal stress is nearly normal to that face, to counteract the large normal pressure of the water. Mr. Cain.

The same general results were first found theoretically by Mr. Ernest Prescot Hill,* for a dam with the up-stream face vertical, and later by the writer† for the up-stream face battered; both writers adopting the usual approximate trapezoid law for the distribution of the vertical components of stress on a horizontal section.

Fig. 14 represents the changes in direction of the greater principal stresses, as computed by the writer on a horizontal section of a dam, 200 ft. below the water level, the reservoir being full. For reservoir empty, the principal stress at the up-stream face would act parallel to that face.

Now, suppose that, for a dam of the same size and shape, but of less specific gravity, and capable of exerting tension, the resultant on the horizontal section passes much nearer the outer toe or beyond it; then a part of the section would be in tension, but the theory quoted shows that the continuity of stress, in amount and direction, would be preserved, and that there would be no abrupt changes in either, in going from the up-stream to the down-stream face. This continuity is preserved, as the center of pressure on the base moves indefinitely beyond the outer toe, and hence it is true at the limit, when the weight of the dam is neglected and the horizontal section is subjected to a pure bending stress and shear, due to the water pressure alone. Exactly the same reasoning applies to the toe, Fig. 1, supposed to be homogeneous, if we combine with the upward acting soil pressure, the friction between the toe and the earth on which it rests. The resultant of this frictional resistance, acting to the right, and the vertical soil resistance, meets the section, $I\ N$, or $I\ N$ produced; giving, as in the case of the dam, continuity of stress throughout the section. Now, let the friction force tend toward zero, then the point where the resultant meets $I\ N$ produced recedes indefinitely from I, but the continuity of stress, in amount and direction, is always preserved, and hence it is preserved at the limit, where the friction force is zero and there is only bending stress and vertical shear on the section, $I\ N$.

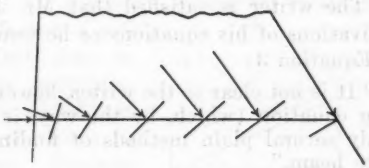

FIG. 14.

Thus, it has been rigorously proved that the bending stress in the toe, Fig. 1, whether or not soil friction is included, is inclined at the angle, β, to the horizontal at N, and this inclination is gradually and continuously lessened as we proceed down the section, $N\ I$, to I, where

*In a paper on "Stresses in Masonry Dams," *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 134.

†*Transactions*, Am. Soc. C. E., Vol. LXIV, p. 208.

Mr. Cain.

it is zero. The same result, as to the continuity of stress, holds for any kind of a beam, subjected to any kind of distributed forces. This continuity of stress, in amount and direction, or the gradual change in both in passing along a joint, as proved above, seems to the writer to be axiomatic without any proof; but others may not think so, hence the proof.

It is difficult to see on what basis Mr. Janni assumes the discontinuity of 90° in the direction of the stress in Fig. 9. Presumably, the stresses act in the direction of the dotted lines. An obvious remark is, that if such directions were the true ones, then the arms of the corresponding stresses are less than those corresponding to the writer's hypothesis; so that even the writer's formulas would be on the danger side for the distribution of stress shown in Fig. 9; and the ordinary formulas, which Mr. Janni advocates, would be still farther on the danger side. There is thus an inconsistency which is fatal to Mr. Janni's contention.

The writer is satisfied that Mr. Janni has not read carefully the derivations of his equations or he would not have stated with reference to Equation 3:

"It is not clear to the writer, however, why the author gives a rather long equation (which, by the way, is not correct), when there are already several plain methods of finding that axis in a reinforced concrete beam."

The reference is to the neutral axis. It is hardly necessary to remark that if the "plain methods" are those pertaining to a prismatic beam (the only case ordinarily treated), they are entirely inapplicable to a wedge-shaped beam.

From overlooking this fact, Mr. Janni's Equation 23 is in error when β is not zero. The correct formula, assuming parallel compressive stresses, is given by Equation 15.

$$f_c = \frac{2\,M}{(b\,d)^2\,(k\,j)\,\cos.^2\beta}.$$

By the aid of Table 1, the values of f_c can be computed for various values of p and β, when b, d, and M are given.

In all the formulas, the value of β was supposed not to exceed about 45°, for reasons given, so that, when Mr. Janni, in Fig. 10, makes $\beta = 90°$, he is simply erecting a "man of straw" of his own in order to knock him down.

Further, in Fig. 10, to show a wedge-shaped beam, any dotted line must be extended to the left to represent the new upper surface, making the angle, β, with the horizontal. We are thus led to Fig. 1, where the resistance to bending along a vertical section is decreased, as compared with the case where $\beta = 0$, because the compressive stresses are now inclined to the horizontal and thus have shorter arms than for the case of the prismatic beam.

Mr. Cain.

The writer is pleased to know that Mr. Mensch thinks that the "assumption that the compressive forces are parallel to the compression face is certainly permissible in all cases where the percentage of reinforcement is low, say, less than 1 per cent." The next remark, that it "errs only slightly on the side of danger", presumably refers to the hypothesis of conservation of plane sections being adopted, as in the usual theory. Some experiments of A. M. Talbot, M. Am. Soc. C. E., seem to substantiate the hypothesis, those of Professor Schüle going the other way. For working loads, the hypothesis is universally used, as it is found to give safe results for reinforced prismatic beams, hence its adoption by the writer for the wedge-shaped beam, with the distinct proviso, however, that it was not to be used for large values of β.

Not only on this account, but principally because it was thought that the hypothesis of the compressive stresses, all acting parallel to the face in compression, departed too much from the truth for large angles, β was limited to about 45°, though, possibly, the limit might be extended to $\beta = 60°$, if the results are tempered with good judgment, which means a slightly increased factor of safety.

Mr. Mensch refers to the fact that in trusses with inclined compression members, the right section is used. It may be added that the stresses are taken parallel to the faces in metal beams with inclined flanges. Suppose a reinforced concrete T-beam, Fig. 15, with an inclined flange; would not a designer naturally take the compressive stresses in the flange as acting parallel to its surface? It seems absurd to take them as horizontal, particularly for the part of the flange on each side of the stem.

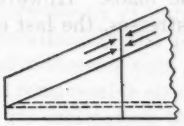

FIG. 15.

It is but one step now to the beam, Fig. 1, without a flange.

Mr. Mensch alludes to the fact that errors are sometimes made by using the factor, cos. β, in place of cos.^{2} β, in finding the compressive stresses. To emphasize this point, it should be remembered that A represents the area of a steel bar, taken at right angles to its axis, so that if f_s is the unit stress, the total stress in the bar is $f_s A$; but, considering a cylindrical "fiber" of the concrete, $N N'$, Fig. 2, the area of its section made by the plane, $I N$, is called Δa, so that the area of a section at right angles to the axis of the fiber is $(\Delta a \cos. \beta)$, and the stress on the fiber is thus, $f_c \cdot \Delta \alpha \cos. \beta$, f_c being the unit stress on the normal section.

For the fiber, $P P'$, Fig. 2, the stress is, similarly, $\frac{f_c}{kd_1} v \cdot \Delta a \cos. \beta$, where $O P = v$, and the sum of such stresses over the area, $O N$, is.

$$C = \frac{f_c}{kd_1} \cos. \beta \sum_0^{kd_1} (v \Delta a) = \frac{f_c}{kd_1} \cos. \beta \frac{1}{2} b (kd_1)^2,$$

Mr. Cain.

since $\sum_0^{kd_1} (v \Delta a)$ is the statical moment of the area under compression about the neutral axis.

Therefore $$C = \frac{1}{2} f_c \, b \, . \, kd_1 \cos. \beta,$$

and the moment of C about G is,

$$M_c = \frac{1}{2} f_c \, b \, . \, k d_1 \cos.^2 \beta \, . \, D \, G \ldots\ldots\ldots\ldots\ldots (8)$$

For the special case given by Fig. 7, G coincides with I, and $D \, G = jd$, on putting $d_1 = d$. Therefore,

$$M_c = \frac{1}{2} f_c \, (kj) \, bd^2 \cos.^2 \beta = \frac{1}{2} f_c \, (kj) b \, p'^2,$$

where $p' = d \cos. \beta =$ the perpendicular from I on $N \, N'$ produced.

Mr. Mensch states that he has advised calculating a wedge-shaped beam, Fig. 11, for a section, $R \, R$. It is not clear exactly what Mr. Mensch means by the statement, for there are mathematical difficulties in the way where the section, $R \, R$, is not parallel to the line of action of the loads. However, accepting the hypothesis of parallel compressive stresses, the last equation gives the precise result,

$$M_c = \frac{1}{2} f_c \, (k \, j) \, b \, . \, \overline{R \, R}^2$$

as $p' = R \, R$, of Fig. 11.

It is to be observed, however, that k and $j = 1 - \frac{1}{3} k$, must be determined for a section (say a vertical section in Fig. 11) through the lower R, parallel to the loads and not for the section, $R \, R$, as follows from the derivation of the formula.

Formula 16 is in the simplest possible form, and by aid of Fig. 6, the values of k can be read nearly to thousandths, and the computation easily effected. As a matter of fact, the steel percentage for the toes, heels, or counterforts of retaining walls, is nearly always very low, so that the steel is the determining factor. In any case, both M_s and M_c should be computed from Equations 14 and 16, for assumed values of f_s and f_c, and the least of the two equated to M, the moment of the external forces; or, if possible, the steel section should be revised, because under-reinforcement is in most cases preferable to over-reinforcement, as it leads to a more progressive, and not a sudden, failure, in case of excess loading.

Mr. Nishkian raises an interesting point in connection with Fig. 4 (referring to the case of the heel-slab with the fillet), that the vertical component of the stress in the inclined rods, for the width, b, should be subtracted from the total external shear to give the shear in the concrete in section, $I \, N$. It may be observed, in reinforced concrete beams, that the bending of the beam entails the interaction of

steel and concrete, without which the steel would be under no stress whatever. Thus the tensile stress in any bar, at a section, is necessarily accompanied with a reaction or an equal bond shearing stress in the concrete, along the entire surface of contact of the bar, to one side of the section. The steel elongates under stress, and this elongation is resisted by the concrete and the total reaction, for the length of bar to one side of the section—the total bond stress—is exactly equal to the tension in the bar. Thus the stress in the bar does not relieve the concrete of shear directly. Even where there are hair cracks on the tensile side of the beam, the full bond stress is exerted between the cracks, giving rise to ordinary shears, and these in turn to indirect tensions and compressions in the parts intact, which play an indispensable rôle in transmitting stresses and maintaining the integrity of the beam. Mr. Cain.

To deduce a formula for shear, only a part of the beam, such as is represented in Fig. 8, is considered. In an actual beam, as repeatedly pointed out, the compressive stresses are not all parallel to the face in compression, but their inclinations to the normal to $I\ N$ become less as the neutral axis is approached. In Fig. 8, the hypothesis of parallel compressive stresses is only used to establish approximately the point D, the point where the resultant, C, of the compressive stresses cuts $I\ N$. If C is now thought of as the resultant of the actual compressive stresses, having varying inclinations to $I\ N$, and calling β' its inclination to the normal to $I\ N$, the formulas which lead to Equation 17 are unchanged, provided we substitute β' for β. The angle does not appear in the final result. To realize further the conditions in an actual beam, tensile stresses in the concrete must be supposed between the neutral axis and the point where the hair cracks are experienced. Although, in the ordinary theory, such tensile stresses are ignored, yet it is seen, because they actually exist, that the section for maximum shear should strictly be taken at O, or at the neutral axis. Such ignored tensile stresses, though small, are nevertheless effective in changing the directions of the stresses in the concrete between the neutral axis and the point where cracks appear.

Formula 17 can be deduced in a different way from that given, which will introduce that interaction between the steel and concrete first alluded to. Let it be assumed that only shear of intensity, v, is exerted on a plane just below O, Fig. 8, that is, perpendicular to $I\ N$. Hence, adopting the previous notation pertaining to Fig. 8, the total shear on the plane is $v \cdot b \cdot dx$.

The bond stress exerted at the surface of contact of any bar, as $I\ I'$, throughout the length, $I\ I'$, is equal to the difference in the tensile stresses in the bar at I and I' and it acts in the direction, $I'\ I$. Hence the total bond stress exerted by all the bars is $(T' - T)$ in amount, and it acts in the opposite direction to T', if $T' > T$.

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If we call β_0 the angle made by the resultant, T (or T'), with the normal to $I\ N$, then $(T' - T)\ \cos.\ \beta_0 =$ the horizontal component of the total bond shearing stress exerted by all the bars, and this must equal the shearing stress, $v\ .\ b\ .\ dx$, exerted on the plane perpendicular to $I\ N$, just below O. Therefore,

$$v\ .\ b\ .\ dx = (T' - T)\ \cos.\ \beta_0.$$

Next, considering the free body, $N\ I\ I'\ N'$, held in equilibrium by the forces, V, V', C, C', T, T', acting as shown in Fig. 8, and taking moments about D,

$$(T' - T)\ \cos.\ \beta_0\ .\ \overline{D\ G} = V'\ .\ dx.$$

Eliminating $(T' - T)\ \cos.\ \beta_0$ between these two equations, dividing by dx, taking the limit as dx tends toward zero and V' tends toward V, and solving for v, we derive,

$$v = \frac{V}{b\ .\ \overline{D\ G}},$$

or Equation 17, which was first derived by a slightly different method.

At first sight, it might appear that the plane just below O, on which there is only shear, should be parallel to $T\ T'$, so that the shear on it should exactly equal the bond stress $(T' - T)$, which must be transmitted by ordinary shear to the neutral axis; but when we consider that along $O\ S$ (by the hypothesis), there is no bending stress, and hence no component of bending stress normal to $O\ S$, it follows that there can be only shear on $O\ S$, and hence a shear only of equal intensity on a plane normal to $I\ N$, say just below O, or between the neutral axis and the steel, if we ignore all tensile stresses in the concrete. The first equation above is thus justified. In the final equation for v, the whole external shear, V, appears, and there seems to be no justification for diminishing it by the component parallel to $I\ N$ of the pull in the rods.

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PAPERS AND DISCUSSIONS

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A STUDY OF ECONOMIC CONDUIT LOCATION.

Discussion.*

By C. E. Hickok, Assoc. M. Am. Soc. C. E.†

Mr. Hickok.

C. E. Hickok, Assoc. M. Am. Soc. C. E. (by letter).—The writer wishes to confirm Mr. Hawgood's statement that the study of this particular conduit comparison was made under the latter's instruction. However, the diagram, Fig. 2, was originated and evolved by the writer.

The writer did not credit tunnels with any saving in seepage, as he was unable to ascertain even an approximate figure for this saving. He agrees with Mr. Hawgood that there would be a saving, but this depends so much on the nature of the tunnel material, which may vary from a pervious soil to hard rock, that there was a hesitancy as to what value to place thereon. There might even be an addition to the discharge of the conduit in the tunnel due to underground waters.

Mr. Hawgood's diagram, Fig. 3, is interesting and is certainly more compact than that of the writer.

In regard to Mr. Stein's remarks as to the thickness of the concrete in the different types of conduit, it should be stated that these drawings, Fig. 1, were not inserted in the paper to illustrate construction details, but simply for pictorial purposes. The drawings were not necessary, and possibly should have been omitted. The writer hopes that no engineer would attempt to construct, for instance, a concrete flume similar to that shown on Fig. 1, without reinforcement. In regard to the 3-in. thickness of concrete lining in the tunnel, the section was supposed to be in solid rock, and no concrete lining was intended to be shown above the springing line. This escaped the writer's notice, due to the fact that he was more interested in the real subject than in construction details.

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* Continued from February, 1914, *Proceedings*.

† Author's closure.

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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

BY MESSRS. CLARENCE S. JARVIS, AND GARDNER S. WILLIAMS.

CLARENCE S. JARVIS, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Lyman's paper, and the discussions regarding stream measurement, are of unusual interest, and will be of much practical value, especially in the irrigated districts. Mr. Jarvis.

Mr. Lyman has admirably disposed of the stupendous task of correlating a mass of data collected under a wide range of conditions, with varying viscosity of the water due to temperature changes and to chemical elements held in solution or suspension, the slight variations in the intensity of the force of gravity, and the personal equations of the observers, as disturbing factors.

The graphical comparison, portrayed so clearly on Plate LXXXI, of actual discharges measured by eminent experimenters with the quantities derived by the use of the diagrams for sharp-crested rectangular weirs without end contractions, proves to be a very satisfactory test of the curves on Plate LXXX.

If a single type of weir were to be adopted, either the Cippoletti or the sharp-crested rectangular weir without end contractions would doubtless serve best in irrigation practice.

One advantage of the Cippoletti weir is illustrated by Robert S. Stockton, M. Am. Soc. C. E., in discussing the "Management of Irrigation Systems".† A portable sheet-steel Cippoletti weir, of 24-in. crest, is used by the ditch rider to measure the water supply in the various small laterals.

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*Continued from February, 1914, *Proceedings*.

† *Engineering and Contracting*, January 28th, 1914.

Mr. Jarvis. In many irrigation districts where the slope of the country is slight, the writer has observed the need for a combination of measuring flume for current-meter gaugings and the approach to a suppressed weir. During the period of high water the canal or lateral is taxed to its capacity, and the obstruction required to give a free fall for such a stream over a weir is prohibitive. During this period current-meter gaugings are in order; and the rating curves for successive years need not change materially.

With the beginning of the low-water period, which, in the Great Basin region, is generally early in July, the duty of water is often increased two- or three-fold, and, with the reduced stream in the canal, it is possible to insert the weir crest in the measuring flume, thus providing a free overfall. The sediment does not interfere seriously after the high water has ceased; and the increased accuracy and reduced labor involved in the weir measurements recommend this method. Under these conditions the rectangular weir without end contractions provides the maximum width of crest and the minimum obstruction to the flow in the canal, and is much to be preferred.

In the writer's opinion, the data contained in Mr. Lyman's paper, and in the discussions, fill a long felt demand, and should wield a strong influence in the direction of higher duty for irrigation water, which will result from its careful distribution and judicious application on the irrigable land.

Mr. Williams. GARDNER S. WILLIAMS, M. AM. SOC. C. E. (by letter).—This paper brings to the front once more the measurement of water over weirs, and embodies the results arrived at by the author in a study of most of the data available in 1905, supplemented by some investigations of his own of more recent date. It is to be regretted that the hitherto unpublished data of experiments involved in this paper were not presented in such a concise form as to have warranted publication. In physical investigation, the one thing above all others that is valuable is a correct record of the experiments and their results. Opinions, theories, and conclusions are likely to change as time goes on, but physical facts remain physical facts, and may be as valuable 100 years hence as they are now, though the experimenter's conclusions may be long discarded.

A considerable part of the experimental work discussed in the paper was performed in the Hydraulic Laboratory of Cornell University under the writer's direction, and much of it under his personal supervision and with his participation, and the data have received careful study at his hands independently of the author's work.

Our basic knowledge of the quantity of water flowing over weirs rests on the experiments of Francis, Fteley and Stearns, and Bazin, all of whom experimented on weirs of different dimensions, and read the head of water at different places or by different devices. No volumetric

Mr. Williams.

measurements were made at Cornell prior to 1905, except in a few low head experiments wherein the inaccuracy of the volumetric measurement, on account of the large area and small rise in the measuring canal, was greater than that of any of the weir formulas could possibly have been. Since that date volumetric measurements have been made at Cornell over a weir 6 ft. long, and at the University of Utah, as described by the author. It is to be hoped that the data of the Cornell experiments may be presented in the discussion of this paper, as the volumetric determinations were probably more accurately made than in any other series of investigations by reason of the measuring basin being a vertical tube or stand-pipe 6 ft. in diameter and about 60 ft. high.

The builders of the Hydraulic Laboratory at Cornell made no provision for the reading of heads on the weirs contemplated. To cut chambers in the solid concrete of the walls and the rock adjoining it involved considerable expense, and when the first weir investigation was called for, that of the U. S. Board of Engineers on Deep Waterways, after some discussion, a device for communicating the head to glass tube gauges was adopted, consisting of a 1-in. galvanized-iron pipe, perforated with ¼-in. holes every 6 in. throughout its length, and placed transversely across the channel of approach, with the openings on the under side, and the pipe 8 in. above the bottom. Three such pipes were set for each weir, varying in distances from the base of the respective weirs. For this device there was then the unquestioned precedent of the similar apparatus at the Holyoke testing flume, which had been used there for many years, and is still. Before this series of experiments was completed, however, the possibility of an erroneous indication of head suggested itself, and, to test the matter, similar pipes were set in the floor of the timber flume leading to the lower weir, with the openings on the top and just flush with the bottom of the channel. The result was somewhat startling, for it was found that, for heads of about 4.50 ft., the original device was giving an indication about 0.30 ft. lower than that shown by the openings in the bottom of the channel. A series of simultaneous observations on the two devices was made, from which the former were corrected to the latter, and it was assumed that this latter device corresponded in its indication to those used by Francis, Fteley and Stearns, and Bazin. At the upper weir it was not so easy to get a series of openings flush with the bottom, as that would have necessitated cutting into the concrete, and after some study of existing data, the so-called standard tube-piezometer, described by the author on page 1577,* was adopted, and on the strength of the experience of Desmond FitzGerald, Past-President, Am. Soc. C. E., with a perforated pipe in a water main,† it was as-

* *Proceedings*, Am. Soc. C. E., for September, 1913.

† *Transactions*, Am. Soc. C. E., Vol. XXXV.

Mr. Williams.

sumed that the indication of this device corresponded to that of an orifice at the bottom of the side-wall similar to the one used by Bazin. With the correctness of this assumption, the writer was never entirely satisfied, but, during his connection with the Laboratory, from the conclusion of the first series of investigations, this device was used at the upper weir in all experiments and very carefully observed, all other devices being compared with it. During the earlier work the upper weir was taken as the standard, and the discharge over it was computed by Bazin's formula, using the indication of the standard tube for the head. In the spring of 1901 the lower standard weir was installed, and the head on it was usually read by the 13-ft., three-tube-piezometer, described by the author on page 1585.* The indication of this apparatus was never considered quite as reliable as that of the standard tube because of a greater likelihood of the presence of air in the pipe connecting it to its gauge. The lower sharp-edged weir was designed to be as nearly as possible a duplicate of the 19-ft., Farm Pond weir of Fteley and Stearns, it being 6.65 ft. high, or $\frac{1}{10}$ ft. higher than the Farm Pond weir, and 16 ft. long. During an extended series of experiments the head on this weir was read at a point 6 ft. up stream from the crest and just below its level, this coinciding with the practice of both Francis and Fteley and Stearns.

To those interested in the more precise measurement of water, the data obtained from the resulting use of different devices for reading head make the most valuable part of this paper, and are embodied in Tables A and B.

The first investigations, as to the effect on the apparent discharges, by different methods of reading the head on weirs, were made by the late James B. Francis, Past-President, Am. Soc. C. E., and are presented in "The Lowell Hydraulic Experiments".† These experiments are discussed by Messrs. Fteley and Stearns.‡

The next investigation of the subject of reading heads, so far as the writer has discovered, was that of Hiram F. Mills, Hon. M. Am. Soc. C. E.,§ which appeared to demonstrate that an opening in the wall of a channel, in order to transmit a pressure corresponding to the height of the surface of flowing water opposite, must be flush with the side, and the connecting pipe must be at right angles thereto.

The subject of indications of head, particularly as applied to closed channels, received further discussion, but without experimental evidence, in the paper on "Experiments Relating to Hydraulics of Fire Streams" by John R. Freeman, M. Am. Soc. C. E.‖

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* *Proceedings*, Am. Soc. C. E., for September, 1913.

† Edition of 1883, page 137 *et seq.*

‡ *Transactions*, Am. Soc. C. E., Vol. XII.

§ *Proceedings*, American Academy of Arts and Sciences, Vol. VI, New Series. Boston, 1879.

‖ *Transactions*, Am. Soc. C. E., Vol. XXI.

Mr. Williams.

In the paper on "Flow of Water in 48-In. Pipes" by Desmond FitzGerald, Past-President, Am. Soc. C. E.,* a statement appears covering the similarity of indication of a perforated pipe laid longitudinally on the bottom of a water main, and a series of circumferential orifices communicating to a chamber surrounding the pipe; and some further investigations along similar lines are presented in the paper on "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah".† Beyond this, the effect of different methods of reading head does not appear to have attracted special notice until the writer called attention to it and presented experimental data bearing thereon, in the discussion of the paper, "On the Flow of Water over Dams," by the late George W. Rafter, M. Am. Soc. C. E.,‡ since which time nothing further has been presented in the publications of this Society, at least as applied to weirs, until the paper under discussion.

In these days, particularly in water-power work, where the competition between various manufacturers leads to the proposal of bonuses and forfeits, based on as small variations of discharge as one-tenth of 1%, exact knowledge of the influence of every departure from the standard of reference is of inestimable importance. In a recent test of water-power apparatus, a bonus of several thousands of dollars was apparently won, a considerable part of which was due to the introduction of irregular methods of observing the head on a weir, without a consideration of their influences.

Theoretically, what is desired in reading the head on a weir is to locate the elevation of the surface of the water, but, with channels of even very moderate widths, this will not be the same at all points across the stream, so that the measurement of it by even several tapes in a cross-section may not give an exact indication of the average head, and if it would, in no experimental investigation wherein the discharge of large weirs has been determined volumetrically, except those at the University of Utah, has such a process been used. The methods of transmitting the head through orifices in the side-walls or in plane surfaces parallel to them, as adopted by the leading investigators, is open to the criticism that the pressure thus transmitted is influenced by the velocity past the orifice to an extent that becomes decidedly appreciable at high heads. If, in order to avoid high heads, long weirs are used, the question then arises as to the applicability of an indication transmitted through the side-wall, to the establishment of the average head across a stream, say, 100 ft. wide. Bazin apparently conceived that the head should be transmitted from a location where the velocity would be the lowest possible, and hence selected a point well up stream from the pressure angle and at the bottom of the

* *Transactions*, Am. Soc. C. E., Vol. XXXV.
† *Transactions*, Am. Soc. C. E., Vol. XLIV.
‡ *Transactions*, Am. Soc. C. E., Vol. XLIV.

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side-wall. For long weirs it seems to be necessary to supplement this orifice by openings in the bottom of the channel at frequent intervals across it, and perhaps the most satisfactory device is a chamber extending across the bottom of the channel and communicating with it through medium-sized orifices at frequent intervals. Although it may be assumed that the indication of head transmitted through such a device cannot be very different, for short weirs, from that used by Bazin, it is nevertheless quite certain that it will not exactly coincide.

Different forms of transverse and longitudinal pipes, variously perforated, are frequently used, but both devices, if outside the pressure angle, are sure to read lower than an orifice in the wall near Bazin's location, on account of the reduction of pressure due to increase of velocity, as the location departs from the wall and bottom. In the experiments presented by the author, the standard tube, although intended, and originally supposed, to measure a head corresponding to that indicated by Bazin's device, fails to do so, and actually gives a lower one. The 11-ft., wall-piezometer coincides more nearly with a Bazin indication, but is still, in all probability, too low, as by the author's equation the Bazin reading is more than 5% higher than the 15-ft. tape.

Being taken in the open channel, the tape readings are far from satisfactory for fine work. At low heads, with smooth water, the tendency of the observer is to cut the surface too deeply, thus giving a low head, and, for high heads and rough water, the observer invariably reads too high. The same applies to observations with the point-gauge in running water. This accounts in part for the irregularity of the lines representing tape readings. Although, if the water is stilled, a skilled observer can do excellent work with a tape or any other form of point-gauge, the ordinary student, dealing with the rough water encountered at high heads, comes very far from getting ideally consistent results.

In order that the record may be complete to date, the writer presents the following data of comparisons of different methods of reading heads.

I.—At Cornell University Hydraulic Laboratory, 1899. Upper Standard Weir, 16 ft. long.

A = Standard tube, by double-tube gauge, as described by the author.

B = Upper transverse pipe, by double-tube gauge.

This was a 1-in. pipe perforated on the bottom with ¼-in. holes, 6 in. apart, extending across the canal parallel to the weir, 8 in. above the bottom and 27 ft. up stream from the 13.13-ft., high, standard weir.

C = Middle transverse pipe, by double-tube gauge.

This was in all respects similar to B, except that its location was 10 ft. up stream from the weir.

A and B and B and C were observed simultaneously on the same scale. Heads are from mean curves of the observations.

Heads.		
A	*B*	*C*
0.164 ft.	0.164 ft.	0.164 ft.
0.492 "	0.484 "	0.492 "
0.984 "	0.970 "	0.985 "
1.476 "	1.461 "	1.495 "
1.968 "	1.938 "	1.993 "
2.460 "	2.410 "	2.489 "
2.950 "	2.860 "	2.965 "

In these experiments it appears that *C* was probably within the pressure angle.

II.—At Cornell University Hydraulic Laboratory, 1899. Lower, Sharp-Edged Weir, 6.56 ft. long.

D = Flush pipe, by double-tube gauge.

This was a 1-in. pipe perforated on top with ¼-in. holes, 6 in. apart, and set with its top flush with the bottom of the flume, parallel to and 37 ft. up stream from the 5.2-ft., high, sharp-edged weir.

E = Upper transverse pipe, by double-tube gauge.

This was similar to *B* and 37 ft. up stream from the weir.

F = Middle transverse pipe, by double-tube gauge.

This was similar to *B* and 19 ft. up stream from the weir.

G = Middle flush pipe, by double-tube gauge.

This was similar to *D* and 19 ft. up stream from the weir.

D and *E*, *E* and *F*, and *D* and *G*, were observed simultaneously on the same scale. The heads for *D*, *E*, and *F* are from mean curves, those for *D* and *G* are from direct observations.

Heads.				
D	*E*	*F*	*D*	*G*
0.920 ft.	0.912 ft.	*F* = *E*	0.744 ft.	0.745 ft.
1.010 "	1.002 "	Nearly.	0.913 "	0.914 "
1.061 "	1.050 "		1.086 "	1.088 "
1.632 "	1.614 "		1.292 "	1.292 "
1.668 "	1.642 "		1.441 "	1.442 "
1.970 "	1.934 "		1.632 "	1.631 "
2.566 "	2.480 "		1.810 "	1.810 "
3.380 "	3.270 "		1.811 "	1.807 "
3.820 "	3.655 "		2.240 "	2.239 "
4.160 "	3.970 "		2.642 "	2.643 "
4.310 "	4.080 "		(3.470) "	(3.460) "
4.820 "	4.560 "		4.270 "	4.277 "
5.040 "	4.720 "		4.670 "	4.681 "
5.450 "	5.090 "			
5.560 "	5.180 "			

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III.—At Cornell University Hydraulic Laboratory, 1899. Lower Experimental Weir, 16 ft. long. U. S. Deep Waterways Section.* 1:1 up stream slope tangent to 3.3-ft. radius curve at top; height, 6 ft.

H = Longitudinal pipe by double-tube gauge.

This was a 1-in. pipe similar to *B*, but laid on the bottom parallel to its axis and about 20 ft. up stream from the weir.

I = Series of ¼-in. orifices, 6 in. apart, throughout the length of the crest, by double-tube gauge.

J = Tape, nailed on the wall vertically from the crest of the weir, and read by contact of water surface.

H, therefore, indicates the head on the weir;
I, the pressure in the sheet at the crest; and
J, the vertical depth of water at the crest.

HEADS.

H	*I*	*J*
0.848 ft.	0.525 ft.	0.582 ft.
1.481 "	0.848 "	1.035 "
2.170 "	1.099 "	1.540 "
2.753 "	1.208 "	2.010 "
3.180 "	1.242 "	2.304 "

IV.—At the Holyoke Testing Flume, 1900. Sharp-Edged Weir, 20 ft. long.

K = Francis orifice, by double-tube gauge.

This was a ¼-in. opening flush with side-wall of canal, 6 ft. up stream from, and 0.34 ft. below, the level of the crest of a weir 5.85 ft. high.

L = Holyoke pipe, by double-tube gauge.

This was a 1-in. brass pipe perforated on its under side with ⅛-in. holes every 2 in., and set 1 ft. above the bottom of the channel, parallel to and 10.2 ft. up stream from the 5.85-ft. high weir.

K and *L* were observed simultaneously on the same scale.

HEADS.

K	*L*
0.392 ft.	0.392 ft.
1.092 "	1.087 "
1.358 "	1.352 "
1.548 "	1.536 "
1.721 "	1.710 "
1.873 "	1.857 "
1.994 "	1.971 "
2.079 "	2.055 "

* *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 284.

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V.—At the University of Michigan, 1912. Special weir, 1.865 ft. long, with one end contraction of 1.08 ft. on side next orifices and gauge-reading devices. Weir, 11.46 ft. high.

M = Francis orifice, by hook-gauge.

This was ¼-in. in diameter, flush with side-wall, 6 ft. up stream from, and 0.34 ft. below, the level of the crest, connected by ¾-in. pipe to a gauge chamber 15 in. square.

N = Bazin orifice, by hook-gauge.

This was 4-in. in diameter, 7.32 ft. up stream from the weir, with its center 0.4 ft. above the bottom of the channel, connected by 4-in. pipe to a gauge chamber 15 in. square.

P = Point-gauge, its contact with the water surface observed electrically by the flash of a lamp.

This was situated in a 10 by 12-in. rectangular box of 2-in. plank attached to the side-wall above the Bazin orifice; the bottom of the box was fully open and 0.79 ft. above the level of the crest, and its down-stream face was 7.47 ft. up stream from the weir. The point was located 7.82 ft. from the weir, and 0.25 ft. from the wall side of the box.

The reading of the several gauges, corresponding to the crest of the weir, was determined both by wye-leveling and by water levels from a hook-gauge attached to the weir crest.

For M and N the crest reading by the water level was 0.005 ft. higher than by the wye-level, and for P, 0.011 ft. higher. This indicates that the determination of crest readings by wye-leveling results in slightly too high a head being used for the discharges. Similar results were obtained at the Holyoke testing flumes.

The heads observed were:

M	N	P
1.1551 ft.	1.1623 ft.	1.1758 ft.
1.3575 "	1.3883 "	1.3918 "
....	1.4572 "	1.4602 "
1.4386 "	1.4435 "	1.4456 "

It is to be noted that, on account of the small size of the opening for M, the water rose very slowly in the gauge tank, and the low reading of the second, as well as the relatively high reading of the fourth, observation may be partly due to that cause. The comparisons of N and P are believed to be very accurate.

As to the comparisons in the author's Tables A and B, it may be said that at the lower weir those of the Francis, float, and bottom piezometers, when more than one was used, being read by the same observer on the same scale of a double-tube gauge, should have been,

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and in all probability were, very carefully observed, and errors in one would be duplicated in the other.

The same is true as to comparisons at the upper weir between the 6-ft. longitudinal and 6-ft. wall, the 6-ft. longitudinal, the 11-ft. wall, the 2-ft. wall, and the crest piezometers which were observed in pairs on double-tube gauges, and the connections shifted so that the 6-ft. wall and 6-ft. longitudinal were observed with each other, and one of these two with all the others.

Tape and point-gauge readings, the standard tube, Bazin hook, and the longitudinal piezometers at the lower weir were necessarily read by different observers, and their differences involve a personal equation as well as possible inaccuracies in establishing the zeros or crest readings of the scale. The greatest care was taken in observations on the standard tube and the Bazin hook, and the work was very well done on those instruments and may be safely accepted. As to the tapes and the other apparatus, the writer does not feel quite so confident.

Table 0 consists of a series of coefficients to apply to the discharge of broad-crested and irregular weirs of a height of 11.25 ft. to give the discharge of weirs of other heights. This table is based on the effect of changing heights of sharp-crested weirs. Although the data used by the author seem to be the best we have, it is, nevertheless, to be remembered that the effect of a change of height on a broad-crested weir may be less than on a sharp edge, and hence the changes indicated by Table 0 may be too great. In other words, it seems probable that for flat-crest weirs higher than 11.25 ft., the discharge may be greater than that obtained in Table 0, and for similar weirs of less height than 11.25 ft., the discharge may be less than that indicated by the table, both by a small amount.

The evidence of Plate LXXXI, that all sharp-edge weir experiments fall substantially within 3% of a mean curve, is certainly valuable.

The curves shown on Plates LXXXIV and LXXXV are to be considered as approximate near the points where the lines deflect. There are conditions under which abrupt changes of condition arise in the discharge of weirs, but certainly nothing of the sort should appear in the standard weir at the points of deflection of its lines on these plates. More numerous experiments in the region of the deflections would show a gradual, not a sudden, change in the direction of the line.

In discussing the indication of the Francis orifice at the Lower Cornell Weir, on page 1548,* the author calls attention to apparently erratic readings, and offers as an explanation the possibility of an oblique current. This orifice was ¼-in. in diameter and near the center

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of a smooth brass plate, 12 in. long and 6 in. wide, set flush with the timber forming the side of the canal. The likelihood of a current causing the discrepancy is remote, but a much more probable explanation is that the zero of the gauge was very close to the floor, and the observer had to lie in a very cramped position to observe low heads. All readings of the scale below a head of 1 ft. are likely to be low by a small amount, including the determination of the zero, on account of the observer not getting his eye on a level with the water in the glass. The effect of this at low heads would be to give a reading ranging between 0.004 and 0.002 ft. in error, which might be either high or low depending on the relative care with which the crest reading and low-head observations were made. From Plate XCIII it would appear that the low readings were high, which means that the zero was more accurate than the low-head readings.

The author's comparison of these indications by the approximate straight lines is open to some criticism, as the deviation of the line at some points from the true locus may be greater than that of the observations it is intended to harmonize.

On page 1579* attention is called to a note in the Laboratory Record, that the 6-ft. tape at the upper weir was below the beginning of the surface slope, and, therefore, was reading low. The statement is correct, and all the 6-ft. piezometers were similarly affected, and the fact stated, that the standard tube, 26 ft. up stream from the weir, was giving a lower reading than these, is to be explained by the relatively higher velocity past its orifices than past those near the wall. The reduction of pressure due to velocity past openings was imperfectly appreciated by the writer until experiments in a closed pipe showed that the pressure indicated by an orifice in a surface parallel to the axis decreased from the walls to the center when water was flowing.

In the writer's reduction of the experiments at Cornell, the results of which appear in the volume of Hydraulic Tables, of which he is a joint author,† the various discharge curves were compared with each other, with the sidelight of his personal knowledge of the peculiarities and characteristics of the several observers and apparatus involved. No attempt was made to establish the actual discharge in any case, but only to establish a ratio by which the discharge of a similarly dimensioned sharp-edged weir should be multiplied to give the discharge of the irregular weir. For himself, it involves less labor and conduces to accuracy to take the discharge of a sharp-edged weir from a reliable table and increase it by a known percentage than to compute the discharge *de novo* by using a special coefficient. If others prefer different methods, he has no objection, always bearing

* *Proceedings*, Am. Soc. C. E., for September, 1913.

† "Hydraulic Tables", Williams and Hazen, John Wiley & Sons, New York City.

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in mind, however, that the base discharge of the sharp-edged weir may be somewhat in doubt for cases beyond the range of experiment.

To the preparation of this paper the author has devoted a great amount of time and labor, a fact which no one more fully appreciates than the writer. He has assembled and presented in new form much valuable information, and for this is entitled to the thanks of all interested in the subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

BY MESSRS. V. H. HEWES AND LAZARUS WHITE.

Mr. Hewes.

V. H. HEWES, M. AM. SOC. C. E.—The speaker would be glad if some member of the Board of Water Supply, who is familiar with the grouting work used in the siphons on the line of the Catskill Water Supply for New York City to check the excessive inflow of water during construction, would describe the method used in that work.

The speaker was granted the privilege of visiting the inverted siphon under the Hudson River where a heavy inflow was encountered, which threatened to drown out the pumps, although extra pumping facilities had been provided to care for such an emergency. A bulkhead was placed in the drift near the heading; pipes were introduced through the bulkhead, and grout was pumped through them, thus effectually reducing the inflow. After the bulkhead was removed and the work of drifting was continued, it was found that the cement had filled the rock seams both laterally and longitudinally. A sample of porous rock was shown which had been taken from another siphon on the line, where the grouting method was used to check the inflow. This sample seemed to be thoroughly impregnated with cement.

A description of the method used to relieve the water pressure on the outside of the concrete lining until the concrete had thoroughly set at points where there were small seams in the rock walls, and the subsequent grouting of these seams, would be exceedingly interesting.

Mr. White.

LAZARUS WHITE, ASSOC. M. AM. SOC. C. E.—It is interesting to compare the results at the Estacada Dam with those obtained on the Catskill Aqueduct. The Ashokan and Kensico Dams have very deep concrete cut-offs. Some grouting was done below these cut-offs, but only

* Continued from March, 1914, *Proceedings*.

Mr. White. as a secondary matter. In the sinking of the shafts there was considerable grouting, which afforded a good opportunity to see what it would do, because, in sinking a shaft, the portion grouted is subsequently excavated. The grouting of tunnels during driving also gives very definite information. The grouting work on the Catskill Aqueduct has been going on for 5 or 6 years, and has been watched for this entire period, so that observing engineers have come to some conclusions as to its results. These conclusions are remarkably similar to those expressed in this paper, which is exceptionally instructive.

The lack of success of a portion of the Estacada grouting is frankly admitted. It seems that in the last few years grouting has been made a sort of fetish; that is, people have been expecting too much from it. At best it is an uncertain proposition, and should be secondary to the customary work. The same applies to the excavation of tunnels. More water is cut off by the concrete lining than by the grouting. Grouting at times is wonderfully successful, but the conditions then are rather favorable; where the rock is hard, with definite splits and channels, the grout can be forced into it readily, but where it resembles the volcanic rock described by Mr. Rands the grout cannot be forced in thoroughly, but takes its own channels. It may run a long way from the part to be grouted, and on excavating this part, very little of it will be found. No one knows where it went—all that is known is that one has paid for it. At some shafts on the Catskill Aqueduct holes have been repeatedly grouted, and when the excavation was extended through the grouted zone the grout could hardly be seen—the same being even more true of grouted tunnels.

As a general proposition, the grout goes into places which the engineer has good reason to know exist, that is, into definite cracks or openings. At some shafts grouting has been very successful, but as already mentioned, the conditions there were favorable. Where unfavorable conditions were found, for instance, resembling those at the Estacada, grouting was not very effective. Judging by experience on the Catskill Aqueduct, the conclusions of this paper are entirely sound.

Grout always seems to obtain some set, but if it is forced into a place where there is water under pressure, and if the water is circulating, it will carry the grout out of the seams before it has a chance to set. Grout has been chased over tunnel arches, up and down, for hundreds of feet. Before the grout can set, it may be forced out, and pipes and seams which were thoroughly grouted may become empty, but if the grout is kept in place long enough, it will set no matter how much water is put in. This seems to be the speaker's experience.

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AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

Discussion.*

BY MESSRS. E. W. JAMES, ARTHUR H. BLANCHARD, AND SPENCER J. STEWART.

Mr. James.

E. W. JAMES, ASSOC. M. AM. SOC. C. E. (by letter).—The author has been particularly fortunate in his studies of sand-clay mixtures for road surfacing, in that his investigations have been made at the best point in all North America for such a purpose. The region especially concerned in the sand-clay type of construction is the Coastal Plain between the Upper Piedmont and Tidewater. According to State lines, therefore, Georgia, and especially North Georgia, is in the center of the sand-clay territory. The best natural mixtures are probably found wherever the Orangeburg sands of the Lafayette formation are prevalent or common. Having confined his investigations to these conditions, the author's conclusions are doubtless stated a little more strongly than they would have been had his experience or studies covered a wider territory. It is safe to state that sand-clay roads can be built in few sections of the United States in such a way as to give as great a degree of service as in Clarke or Elbert Counties, Georgia.

For several years the writer has had a wide experience in sand-clay work throughout all the Coastal Plain from Virginia to Southern Texas, and recognizes the importance of this type of construction. For a large section of the Southeastern States it is the only possible type of improved road for a large mileage of the country highways. The importation of stone is prohibitive because of excessive cost; shell is too friable and soft; brick, even on a natural sand foundation, is too expensive; and concrete is practically out of the question because of the cost of both sand and aggregate.

* This discussion (of the paper by John C. Koch, Assoc. M. Am. Soc. C. E., published in February, 1914, *Proceedings*, and presented at the meeting of March 4th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. James.

For many counties in the Southern States, therefore, the sand-clay road is the only improved type within the means of the locality, and the only one economically warranted by the conditions of traffic at the present time and probably for many years to come. In many sections, especially in the Tidewater and Coast counties the natural soils are prevailingly sandy. It is difficult to find good clays. The sand beds are as bad or worse than the clay mud of the uplands, and, moreover, they are bad, not only in winter, but all the year round. For this reason the sand-clay road is especially valuable in these sections. The natural soil is well drained, and the latitude precludes any lasting or great degree of frost. The question in these regions is not commonly to select materials that will produce a sand-clay of certain stability, but to use the materials at hand so skilfully as to produce alleviation of the almost intolerable conditions presented by the sandy roads.

In the counties along the coast, and usually for about 100 miles inland, the sands are fine and often floury; all clay, except occasional small pockets of white or gray pipe-clay, or a mottled red and gray variety, carries a large percentage of this fine sand. Consequently, the field methods of testing the clays described by the author would, in the writer's experience, result in rejecting a majority of those locally available, and of discarding all but the rarest deposits of sand.

The author's conclusions, however, are recognized as well founded, especially in regard to the value of coarse sand.

For sand-clay mixtures, the writer has frequently used a most simple field test, which has served his purpose well. The sources of material are located and selection is made, depending usually on the length of haul and the depth of the deposits beneath the surface. The immediate problem is to determine the best mixture of the materials selected.

Typical samples are taken of both sand and clay. Mixtures are made, ranging from 1 part sand to 3 parts clay, up to 3 parts sand to 1 part clay, or sometimes beyond these limits, if the materials appear to warrant it. These mixtures should be made to vary by one-half of 1 part, 1:3, 1:2½, 1:2, etc., 1½:1, 2:1, 2½:1, etc., and should be worked up with water into putty-like masses.

From each test, mix a small sample of from 1 to 2 cu. in., cut out with a small measure. The writer has found a small medicine glass, or even a large brass thimble, handy. It is only essential to get equal samples from each test mix. These samples are then rolled between the palms of the hands into reasonably true spheres and placed in the sun to dry. Some designating marks may be scratched on them. When thoroughly baked, they are placed in a circle in a flat pan or dish, and enough water is poured in the pan to cover them, care being taken not to pour the water on the samples.

Slaking will begin at once. The lapse of time found by Mr. Koch with his compressed specimens is not found at this stage. The slaking, however, will proceed at different rates. The sandy specimens will break down first, those with excessive clay will disintegrate second in order, and those having about the proper proportions will act more slowly. Usually, there will be one or two that determine the proper proportions of the materials, and, in the writer's experience, these will usually lie together in the series of test mixtures. Mr. James.

A supplementary test of some value can also be made on the dry spheres. Lightly rubbed with the thumb, those having too much sand will break down rapidly. Those having too much clay will soon begin to "dust" away, while those having the most stable mixtures will assume a slightly glazed effect under the light rubbing, due to the moisture and oil of the skin. These two tests will not give the same results. The dry test will indicate a mixture richer in clay as the better one, and the wet test will indicate a sandier mixture. The sample indicated as satisfactory under the wet test that lies between the other two will prove best in service.

These extremely simple tests do not determine the sand or clay content of the mixture, for the clay selected almost always contains considerable sand, and this, in most cases, contains silt or clay. The tests serve, however, to fix the actual values in the mixture of the pit run, as represented by the samples.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The speaker's discussion will be limited to calling attention to certain features of the construction of sand-clay roads which it is believed the author should include in his very valuable contribution to the literature on this subject. Mr. Blanchard.

The author states that the first cost averages $500 and the annual cost of maintenance is $5 per mile; and then emphasizes the fact, agreed with by the speaker, that there is a place for the sand-clay road. Very little information, however, is given with reference to local conditions. As the fundamental principle of sound practice is to use that type of road or pavement which is economical and suitable for a given set of conditions, it would be of great value to American engineers to have at hand especially more detailed knowledge relative to the traffic on the various types of roads referred to in the paper.

Another part of the paper which it is believed could be advantageously amplified is that relative to the effect of different percentages of sand-clay mixtures retained on the 10-mesh sieve, when such mixtures are used under various climatic and traffic conditions.

There are many highway engineers who, without doubt, will disagree with the author in dividing gravel and sand on the basis of material retained on or passing the 10-mesh sieve. The speaker believes that the 4-mesh sieve is rapidly being adopted as the basis of division between these materials.

Mr. Stewart.

SPENCER J. STEWART, ASSOC. M. AM. SOC. C. E.—The speaker has read this paper with great interest. From it it is evident that the clay-bearing materials are being given proper economic consideration in the highway development of the South. In New York State, however, clay is a material which, according to many highway engineers, possesses little or no value, but, on the contrary, contributes characteristics highly detrimental to proper highway construction.

Specifications prepared by highway commissioners, especially those of the State of New York, have insisted that the foundation course shall be filled with screenings, gravel, or sand, and have invariably ruled against a filler which would be classified as clay. From the speaker's experience, covering responsible charge of the construction of more than 300 miles of highways, at a total cost of $4 000 000, he has been led to believe that a clay-bearing material, if applied as a filler when dry, makes a foundation as stable, and less likely to disintegrate, than a filler of sand or the screenings of a non-cementitious stone.

It is common knowledge that clay may be used to advantage in binding the top course of macadam roads in cases where the dust from the crushed stone possesses no cementitious quality.

Perhaps it is not inappropriate, and may prove interesting, to call attention to the results obtained from the use of the sand-clay-bearing gravels of the Hudson River as a material for successful road construction. What is true of the cementitious gravel of the Hudson would be true of similar deposits in other parts of the country, but it should not be presumed that every gravel bank necessarily contains the characteristics of a cementitious gravel.

This material has been used for years on the Parkway Systems of Greater New York, originally as a water-bound material and, more recently, covered with hot oil. The roadways proved very satisfactory until the advent of the motor bus, which unusual traffic they could not withstand, nor was it ever intended that they should.

This same material has been used on the sandy soil of Long Island with excellent results, in spite of the crude method, or rather lack of method, used in the construction of some of the highways. Frequently, it was dumped on the sandy road, spread carelessly, and left for the traffic to compact into a smooth surface. In spite of this, however, it soon ironed out and made a most excellent road covering, considering the time and money spent on construction. On the other hand, under the direction of the New York State Highways Commission, roads of the highest type have been built of this material by the so-called water-bound method and the application of hot oil, proper care being exercised as to rolling and puddling.

With respect to roads of this latter class, the speaker wishes to call attention to one partial failure, point out the causes, and perhaps draw a lesson from the experience.

Mr. Stewart.

Hot oil had been applied during the late fall, and properly covered, but, in the following spring, the heat of the sun caused the oil to bleed and adhere to the iron tires of slow-moving vehicles, which drew up part of the top course and deposited it a few feet ahead. This resulted in alternate holes and mounds, and caused a very uneven surface which gradually became more pronounced as the holes became larger through wear. As soon as the first indication of bleeding occurred, the surface should have been covered with sand to prevent this unfortunate condition. One enterprising citizen did apply this remedy in front of his own property, with the result that the top surface is as good to-day as it was at the time of the completion of the road.

Hot oil treatment on water-bound roads, whether of stone or gravel, must have a certain amount of immediate attention during the period when the oil is susceptible to bleeding. Such attention simply means the application of blotting material.

As a foundation course on sandy soils, cementitious gravel has proved an unqualified success, at least in the speaker's experience, for he has known of no failures. Failures do not mean that a stretch of 50 ft. in 10 miles might not prove unsatisfactory; that might happen with any foundation, due to causes which could not be guarded against at the time by the construction engineer. The larger the mineral aggregate, the better the foundation, if just sufficient sand and clay are used to fill the interstices. If a mixing-method top is to be placed, the speaker would advise that the foundation course be thoroughly puddled and dried before the top course is laid.

As a foundation course for country highways, it is believed that gravel containing fines of a cementitious nature is to be preferred to either broken stone or concrete.

First, it costs less. Incidentally, it might be of interest to know that practically all materials entering into the construction of roads on Long Island must be imported, whether it be crushed stone, Hudson River gravel, or so-called Long Island gravel. The latter exists only on the north shore of the Island in any quantity sufficient to be used economically for work of any moment, must be dredged, barged to Long Island City, there transferred to cars, and then hauled to its destination. Even then, this material should only be used in concrete construction.

Where materials have to be imported, it should be remembered that broken stone, even at the same cost at the point of destination, costs approximately one-third more than gravel, due to its consolidation of about 33% under the weight of the roller, and also to the necessity of adding from 25 to 30% of sand or screenings to fill the voids. On the other hand, gravel, when measured in bulk on barges or cars, will about equal the quantity rolled in place on the road, requiring no extra material for filling. Concrete requires about 80% of the

Mr. Stewart.

rolled quantity of stone and gravel, with the addition of 40% of sand, together with the cement, the quantity of which depends on the proportion of the mixture.

On the contrary, where the importation of road-making material is unnecessary, the use cementitious gravel saves the cost of crushing and the incidental expenses associated with such process.

Second, a proper gravel foundation possesses the resiliency which is generally recognized as necessary for a successful road, especially for country highways. Broken-stone foundations possess this characteristic in a less degree, and concrete is entirely lacking in it.

In the case of asphalt pavements, they are now designed to overcome the lack of resiliency of the concrete foundation by the use of an intermediate or cushion course between the concrete and the asphalt surface. In the same manner properly designed mixing-method pavements of small mineral aggregate when placed on concrete foundations, should be provided with this cushion course.

Where conditions of traffic require a pavement of considerable permanency, the mixing method may be used. The speaker designed a pavement consisting of a mixture of asphalt and gravel in the proportion of 1 cu. yd. of loose gravel to an average of 20 gal. of asphalt, the gravel containing not less than 10% of clay. The gravel was bank run, the largest particles of which were 2 in. in the longest dimension, containing sufficient fines to fill the voids partly. The bitumen was a fluxed natural asphalt with a penetration between 10 and 13 mm. when tested for 5 seconds at 77° Fahr., on a No. 2 needle weighing 10 grammes.

The gravel was heated in a mechanical revolving dryer with a temperature of more than 225° Fahr., after which the asphalt, heated to not less than 275° Fahr., was added; then the mixture was placed in a revolving mixer until thoroughly and completely coated with bitumen. The mixture, at not less than 225° Fahr., was spread on the prepared bottom course with shovels from dumping boards and raked to a uniform surface with hot rakes, after which it was rolled with a self-propelled roller weighing at least 10 tons, until it was thoroughly consolidated.

On some sections of this pavement there was placed ½ in. of gravel screenings containing not less than 10% of clay which was saturated with water and rolled thoroughly and continuously until a clay mortar had been obtained. This process filled all the surface interstices with a gritty and adhesive substance which made the road practically "non-skid". In a short time the traffic drove away all surplus screenings, leaving a mosaic surface.

A different method of treatment was applied to this form of pavement in the more thickly settled communities. After the mixture had been rolled, it was covered with a coat of hot oil as a seal coat,

this seal coat being applied in November. In the following spring, under the action of the sun, the road bled to some extent, and became so sticky in a few places that the oil adhered to wagon wheels, which pulled up some of the top course. This condition could have been avoided if the authorities in charge had covered the pavement with sand or gravel screenings as soon as it became apparent that the hot oil had a tendency to bleed. No material harm was done, however, as the continuous traffic carried sufficient sand and dirt on the pavement so that in a short time the stickiness of the oil had disappeared. Mr. Stewart.

Such treatment obviates the general complaint against the so-called stone-mixing method pavement where a greater quantity of asphalt is used and where the seal coat is of the same consistency as the asphalt binder in the top course proper. These objections arise from the hardness and slipperiness of the surface, which, during the greater portion of the year, make it undesirable for horse traffic because of the former characteristic and to motor traffic on account of the latter.

It is suggested that, where the hot oil is omitted at the time of the original construction, the mosaic surface be covered with a hot oil treatment of ½ gal. per sq. yd. during the following year. With the hot oil as a squeegee course, in place of the seal coat of the same consistency of the bitumen, the pavement proper gives a less hard and slippery surface, which is desirable on country highways.

The speaker's experience has convinced him that a large plant, costing from $5 000 to $8 000, is not necessary in order to construct this pavement successfully, and that equally good results can be obtained from the use of small mixers costing from $1 500 to $2 000.

This pavement without the oil treatment cost about 85 cents per sq. yd., and with an oil seal coat about 90 cents per sq. yd. for a pavement 2½ in. in depth. This cost compares favorably with similar figures for mixing-method pavements in other parts of New York State. The speaker quotes from a statement attributed to a superintendent of highways, of New York State, relative to costs of similar pavements of graded stone material, as follows:

1½ in., California asphalt	$1.20	per sq. yd.
2 in., Topeka	1.20	"
2 in., Warrenite	1.30	"
2 in., Bitulithic	1.60	"

The gravel used in this pavement cost approximately $2.35 per cu. yd., f. o. b. destination, but where material can be obtained near the site of the works, the cost of 85 cents per sq. yd. can be materially reduced.

From observations of mixing-method pavements laid in New York City and vicinity, with graded stone as a mineral aggregate, it is ventured that the percentage of disintegration is as great, if not greater, than in the bituminous gravel pavement laid during the same

Mr. Stewart.

period of time. In fact, after 1 year's wear, out of a total of 112 000 sq. yd., or 0.04 of 1%, less than 45 sq. yd. of this gravel mixing-method pavement had disintegrated.

The road which received the treatment of hot oil in a squeegee course was dug up in many places during the year following its completion for investigating purposes, and, where removed, the material thrown back retained its vitality to such an extent that the top course healed itself so that a casual observer could not discover where the pavement had been disturbed.

In designing a country highway, not only the original outlay, but the cost and ease of maintenance should be given proper consideration. Those who have had experience in the upkeep of gravel roads realize the small expense necessary to retain them in their original condition. Where slight depressions occur from time to time, all that is necessary is to place a sufficient quantity of gravel in these depressions and allow the traffic to consolidate it. In cases where the road has become pitted to such an extent that it requires more extensive treatment, it can be scarified, harrowed, re-rolled, and puddled, and, with the addition of a small quantity of material, can be made as good as the original at a comparatively slight cost, due to the presence of the clay-binding material. This treatment cannot be successfully consummated in broken-stone roads, for as soon as the bond of an old stone macadam road is broken, the material is usually worthless. These peculiarities of the two materials result in a very pronounced saving in the maintenance of gravel roads, as compared with that of broken-stone roads, whether or not they had been subjected to hot oil treatment. The ease of maintenance of the gravel, mixing-method pavement is sufficiently illustrated by the self-healing incident referred to.

The speaker is led to believe that gravel, sand, and clay have not been given the consideration they deserve by the highway engineers of the United States. In a section of the country which possesses clay and sand-bearing gravel, engineers can well afford to investigate its uses for road-making purposes, either as a foundation course, a water-bound top course with or without hot oil treatment, or a top course using the mixing method.

To conclude, the speaker cannot express himself more clearly, relative to the use of gravel, than by referring to the following statement:*

"A well built and well maintained gravel road is preferred by automobilists to macadam roads or even roads paved with more expensive materials. The gravel road is springy and resilient and gives good grip to the tires. It does not produce the jar that exists in riding over a paved road with an unyielding foundation."

This is also true of the man who uses his horse for either business or pleasure.

* From *Engineering News*.

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PAPERS AND DISCUSSIONS

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REPORT ON A SERIES OF TESTS ON CONCRETE COLUMNS REINFORCED WITH A SPIRAL OF STEEL.

Discussion.*

BY MESSRS. EDWARD GODFREY, AND A. W. BUEL.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—This is a valuable series of tests. It is unfortunate that such information, and proper interpretation and digestion of the same, were not available before common practice in reinforced concrete design was crystallized into the deplorable shape that it has taken in all building codes and in the report of the Joint Committee on Concrete and Reinforced Concrete, as well as in the standard books on the subject. Mr. Godfrey.

"Some experiments then made on the ultimate strength of columns with longitudinal steel bars embedded in the concrete indicated that such reinforcement, unless hooped or banded at frequent intervals, could not be relied on to increase the strength of the column, and in some cases might even render its strength less."

It is gratifying to know that truths, the recognition of which the writer has been seeking since 1906, are beginning to dawn on the Profession. He predicted that it would take 10 years. It will probably take that before the Joint Committee and the authors of building codes recognize the truth. Books will follow some years later, and the rodded column will have eked out its murderous existence.

The writer has followed this subject pretty closely, and, so far as he is able to learn, no one but himself had condemned the rodded column until recently, when it was criticized severely at a meeting of an engineering society in Chicago. It is absolutely true, as the

* This discussion (of the paper by C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie and C. O. Carey, published in February, 1914, *Proceedings*, and presented at the meeting of March 18th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Godfrey.

authors state, that years ago experiments indicated that longitudinal rods do not reinforce a column, but it is just as true that the very authorities who made these experiments and who discussed them in their books, endeavored to explain away the facts and to apologize for the particular specimens of concrete in the weakened so-called reinforced columns. These same authorities recommend the addition of from 15 to 39% of strength for every 1% of this weakening steel.

What are the facts to-day? Every building code allows rodded columns; practically all the standard books recommend them; the Joint Committee Report allows them; nearly every great wreck has had them, and has been the result of them.

The authors are wise to ignore rodded columns in their tests and investigation, for the rodded column has absolutely nothing to recommend it to any intelligent man, and there can be no rational incentive to make tests on them except for the purpose of demonstrating their unfitness for structures, the unfitness that an intelligent analysis of the column will demonstrate, entirely apart from tests. It is deplorable that, though the Profession has been in possession of this experimental demonstration for a number of years, the facts have been so perverted, and recommendations have been so utterly contrary to what they have shown, that standard methods of constructing columns are criminally bad.

The writer would put quite a different interpretation on the tests made by the authors from that given in their paper. Their comparison between the plain and the hooped column is unfair to the former. Not that the plain column has any right to consideration as a structural member, but such comparisons give the hooped column an apparent advantage over other structural members, in the matter of unit stress, which it does not possess.

In calculating the unit load at first sign of failure, they have taken only the area of the core for columns with hooping, whereas they take the full sectional area of the plain columns. The area of the core is about 10 sq. in. and that of the full section is about 13 sq. in. Making this approximate correction on all their columns having embedded coils, the writer has discovered the following remarkable facts: The average load at first sign of failure for the 24 plain columns is 2 250 lb. per sq. in.; the average load at first sign of failure for the 18 columns having less than 1% of hooping is 2 250 lb. per sq. in.; the average load at first sign of failure for the 28 columns having more than 1% of hooping is 2 230 lb. per sq. in.; and the corresponding average values for the ten highest in the foregoing groups are:

Plain columns, 3 057 lb. per sq. in.
Less than 1% hooping, 2 614 lb. per sq. in.
More than 1% hooping, 2 769 lb. per sq. in.

Mr. Godfrey.

If these facts teach anything at all, they blazon out the truth that hooping has no influence whatever on the first failure of a column, and it is the first failure that is really the ultimate failure. A column is useless after it begins to spall on the surface.

The data presented by the authors on the ultimate strength of these columns are only of academic interest. They are totally erratic, and simply represent the post-mortem strength of columns which have already failed. How long it may take a rattlesnake's tail to cease wagging after its head is crushed, has no bearing on the safety of the persons who were menaced by the live snake. The load-resisting power of a more or less disintegrated mass of concrete encircled by a steel spiral has no bearing on the design of a safe structure, if the first sign of failure is ignored. The authors might as well take one series of columns and label them "A" and use their full area in determining the unit load, and then take another set which have gone through some treatment or other and label them "B", and use only 80% of their area, and then say, when Columns "B" stood exactly the same load as Columns "A", that Columns "B" were 25% better than Columns "A", as to make the comparison that Table 3 shows on its face.

We have it on the authority of Ernest McCullough, M. Am. Soc. C. E., that in a large number of commercial designs he has never found one which excluded the outer shell in the strength of the column. The writer has persistently refused to have anything whatever to do with the checking of rodded column installations, or doubtless he would have discovered the same thing. There is no reason why investigators should omit the outer shell of a column, either in making comparisons or in discovering the strength of columns.

These tests demonstrate several things. One is that the closer the pitch of the spirals, in general, the greater the ultimate strength of the column. This could be reasoned out by analogy. A brick will stand more load per square inch flatwise than on end. A mortar joint is stronger the thinner it is made. A flat disk will stand much more load than a high cylinder of the same diameter. There is a limit, however, beyond which this added strength is of no use. It would be unsafe to have a column in a structure with the shell outside of the coil spalled off. Hence the "first sign of failure" is the function that must govern in safe design. This value, in these tests, runs from 1 400 lb. per sq. in. up, on the full area in compression, on the hooped columns, or 1 800 lb. on the core.

Now what factor of safety will the Pittsburgh Building Code give, for example, allowing as it does from 1 300 to 1 750 lb. per sq. in. of alleged safe load on columns, as shown by T. L. Condron, M. Am. Soc. C. E..* Other building codes are not quite as bad as that of

*Paper before the Western Society of Engineers, Feb. 9th, 1914; *Engineering News*, Feb. 19th, 1914; *Engineering Record*, Feb. 21st, 1914; *Engineering and Contracting*, Feb. 18th, 1914.

Mr. Godfrey.

Pittsburgh, but those of St. Louis and Minneapolis are close seconds. The gravel concrete common in Pittsburgh is not nearly so strong in cylinders as the limestone concrete used by the authors.

In a testing machine, a column might show a certain strength at first failure, and two or three times as much at ultimate failure, but, if it begins to fail in a building, the sway and eccentricities unavoidable in structures will doubtless mean its ultimate failure.

One of the purposes of a factor of safety is to cover imperfections. What would happen in this "safe" Pittsburgh column, if a workman should drop a wooden block into it accidentally, or if there should be a void, filled up later by a trowel full of mortar? Who would be responsible for the possible general wreck of the building, the workman, the designer, or the commercial interests who get up these building codes? Laboratory tests are more carefully made than actual construction, so that this should be taken into account.

These tests demonstrate that there is no definite relation between the percentage of steel reinforcement and even the ultimate strength of the core. This, together with the facts concerning the pitch of the coil, shows clearly, what the writer has for many years contended, that Considère's formula is wrong. That formula, and it is used in the newest building code, which is that of Pittsburgh, would add to the strength of a column having a given pitch by adding to the size of the wire in the pitch, a perfectly absurd proposition, as the wire might already be strong enough to stand the crushing strength of the disk between two coils.

Another important fact brought out by the authors is that the ratio between the moduli of elasticity of steel and concrete is not more than 10. The value, 15, is very commonly used with a resultant degradation of design, where steel rods are considered in compression.

The authors state:

"That the shrinkage of the concrete, causing initial compression in the steel, might be the cause [of greater deformation in hooped columns] is untenable, because the concrete was immersed in water during the entire aging period."

It is presumed that the columns were dry when tested. Tests show that specimens shrink and swell by the mere process of drying and soaking them.

What the Profession needs is to ascertain the combination of spiral and longitudinal reinforcement for a column of given diameter, which will be consistent and properly balanced. This should then be adopted as a standard, and the value of a column should be based on the kind of concrete used, just as in beams or slabs. The longitudinal steel should be stiff enough and the coils should be close enough to hold these rods from buckling. The writer* advocated this in 1906 and worked out such a standard.

*In his book entitled "Concrete."

Mr. Godfrey.

The Profession is not in need of tests so badly as of proper interpretation of those already made. This is not said in disparagement of those made by the authors. They are of great value. What is sorely needed is complete revision of building codes, books, and of the Joint Committee Report. The present standards of reinforced concrete design are dangerous and outrageous.

Mr. Buel.

A. W. BUEL, M. AM. SOC. C. E.—It appears to the speaker that the programme for this series of tests was planned so that results or conclusions of much practical value or reliability could hardly be expected, at least none commensurate with the expenditure.

The slenderness ratio, about 6.75 for the concrete or 7.7 for the coil, was much too small to represent conditions in practice, and should classify the series with compression tests rather than with column tests.

The concrete mixture, 1:2:4, was too rich to use economically with spiral or hoop reinforcement, and the tests do not show the true value of such reinforcement, which is greater with lean concretes. Some indication of this is shown by a comparison of the short- and long-time tests of this series.

The section of the test columns was so small that the results will hardly be depended on as a basis for structural designing, in view of the fact that engineers already have reports of a considerable number of full-sized tests. Some 6 or 8 years ago, when very few reports of full-sized tests were available, such tests as these would have been received as a valuable contribution. A dozen full-sized column tests, carried out on a well-planned programme, would be a more valuable addition to the experimental data on the subject than several hundred tests on small model specimens having dimensions entirely out of proportion to those generally found in practice.

The pitch of the spirals varied from 0.25 to 0.75 in., in columns of the same group. This, and also the variations in age, make it impossible to compare fairly the results from one test column with those from another, to draw reliable conclusions, or, for practical purposes, to realize what the authors describe as the "Aim of the Tests".

The records of "first sign of failure", particularly for the short-time tests, do not agree with tests reported by other experimenters, where the envelope of concrete or cement outside of the spiral or hooping almost invariably began to show signs of failure as soon as the load reached or exceeded the ultimate load of a similar column without reinforcement. In other words, all previous tests have indicated that spiral or hoop reinforcement does not materially raise the point of first sign of failure. Mr. Godfrey has pointed out what seems to be an error in computing the intensity of stress at first sign of failure: that the area of cross-section enclosed by the spiral was used, whereas the area of the entire column, including the concrete outside of the spiral, should have been taken for this computation. For the determin-

Mr. Buel.

ation of the intensity at the ultimate load, the area enclosed by the spiral is correct, because at that time the outside envelope would have been destroyed.

In a number of cases the pitch of the spirals was somewhat higher than that recommended by Considère for maximum efficiency of hooping. This was confined mostly to columns with the smaller percentages of spiral reinforcement.

Most of the eight conclusions drawn by the authors from the results of this series of tests are in substantial agreement with those deduced from previous experiments, and can be considered as a confirmation of the latter.

Conclusion No. 2 gives a value for e, the ratio of the modulus of elasticity of the steel, E_s, to that of the concrete, E_c, which is remarkably low, considering what engineers have been accustomed to use and the results of previous experiments. The speaker would like to ask the authors whether the value of E_c, which they derive from the formula, $600\,000\ f_c \div s_2$, is the true value of E for concrete alone, and is it not possible that it was affected by the spiral reinforcement to some extent? Also, what was the value of E_c for the plain concrete columns without reinforcement? Is not the E_c computed by the authors' formula, the E for the column as a whole, rather than the E for the plain concrete?

Conclusion No. 5, that the spiral reinforcement materially raises the limit of the first sign of failure, will require further experimental demonstration to be fully convincing, in view of the considerable number of determinations which have shown different results.

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PAPERS AND DISCUSSIONS

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THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

Discussion.*

By Messrs. Philip W. Henry and James J. Hill.

Philip W. Henry, M. Am. Soc. C. E.—This paper contains a great deal of valuable information, particularly for those interested in the construction and operation of railways in Latin-America, and the author is to be congratulated on the exhaustive and intelligent manner in which he has treated this important subject. Of special value is his emphasis on the distinction between a "narrow-gauge railway" and a "light railway", which are often used as synonymous terms, although, as the author points out, a "light railway" may be of any gauge. Mr. Henry.

The speaker agrees with Mr. Lavis that the difference in the construction cost between a narrow-gauge and a standard-gauge railway is generally exaggerated. In a country the traffic of which does not admit of the construction of an expensive railway, and where the prevailing gauge is 4 ft. 8½ in. or greater, the speaker believes that it is better to cut the construction cost by raising the maximum grade and curvature rather than by lessening the gauge of the railway. Though the author treats of railways chiefly in the southern part of South America, more particularly Argentina and Southern Brazil, both lying within the temperate zone, where conditions are quite similar to those in the Central Western States, and where traffic will follow railway construction almost as rapidly as it did in those States, it must be remembered that in tropical South America and in Central America traffic does not follow with the same rapidity, and, therefore, the type of railways for those countries may be quite different from that suitable to Argentina and Southern Brazil. Broadly speaking, it is safe to

*This discussion (of the paper by F. Lavis, M. Am. Soc. C. E., published in March, 1914, *Proceedings*, and presented at the meeting of April 1st, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Henry.

assume that in Central America, the West Indies, and in tropical South America, the gross earnings of a new railway for the first few years will not be much greater than operating expenses, let alone interest on the capital invested, and that the development of traffic will be very slow, unless the parties building the railway undertake at the same time to develop traffic. Even in a highly fertile tropical country, it is seldom that the natives, who are well satisfied with their existing life, will make any effort to develop traffic on any considerable scale. The development of tropical agricultural traffic in most cases demands both capital and skill. This applies particularly to the raising of bananas and sugar cane, two of the most profitable tropical products.

If the railway is built in the high altitudes of Peru or Bolivia, not susceptible of agricultural development, it is essential that mineral traffic be developed, which, again, requires large investment of capital. In some cases, the building of a railway may be followed by the investment, on the part of outside capitalists, in the agricultural or mineral resources of the region traversed by the railway; but, generally speaking, if the parties building the railway are also its operators, it is essential for best results that, at the same time they acquire the concession for building the railway, they should take steps for the development of traffic—a point which is generally overlooked. The Governments often recognize this slow development of traffic, either by guaranteeing the bonds of the railway or by paying a cash subsidy approximating, if not equalling, the cost of construction.

The speaker believes that the author has over-estimated, if anything, the additional cost of constructing a standard-gauge railway (4 ft. 8½-in.) over that of a meter-gauge, especially where he adds an extra cost for "Contingencies, Expenses of Promotion, and Interest during Construction," amounting in all to 15%, which, according to his figures, equals $249 per mile on roads of light earthwork and bridging, $430 per mile on roads of medium earthwork and bridging, and $1 359 per mile on roads of heavy earthwork and bridging. It appears to the speaker that this item of general expense is the same, regardless of the gauge of the railway. In other respects, the speaker believes that the author's estimates are, if anything, more than ample. The extra cost of standard-gauge, given by him in detail for the three classes of railways, neglecting the 15% for general items as above, is as shown in Table 28.

To confirm these figures, it is interesting to compare the actual cost of building a railway of light earthwork and bridging in Bolivia, from Viacha to Oruro, 127 miles in length, with which the speaker was connected in 1906-09. This railway was of meter-gauge, roadbed of 5 m., the average cross-section for the entire length showing excavation actually paid for, both in cuts and borrow-pits, amounting to an average depth of 3.7 ft. Assuming that the railway had been of standard-

gauge, instead of meter-gauge, it would have been necessary to handle $6\frac{1}{2}\%$ additional earthwork. It is assumed that the cost of bridges, trestles, and culverts, and of tracklaying and surfacing, would also have been increased by $6\frac{1}{2}\%$, certainly a liberal estimate. The ties, California redwood, costing about $2, delivered on the work, were 7 ft. in length, instead of 8 ft., if standard-gauge had been adopted, making an additional cost of $14\frac{2}{7}\%$ for ties. Table 29 shows the actual cost per mile of the different items affected—in the speaker's opinion—by the change in gauge, and the additional cost for each item in case standard-gauge had been adopted. Mr. Henry.

TABLE 28.

	Light earthwork and bridging.	Medium earthwork and bridging.	Heavy earthwork and bridging.
Earthwork	$310	$980	$5 010
Bridges	150	500	2 000
Culverts	150	300	1 000
Ties	770	770	770
Laying and surfacing	100	100	100
Yards and sidings	170	170	170
Cattle guards, fences, etc.	10	10	10
Total for construction	$1 660	$2 830	$9 060
Less saving in equipment	100	100	100
Net Total	$1 560	$2 730	$8 960

TABLE 29.

	Actual cost, meter-gauge.	Estimated additional cost, standard-gauge.	Total estimated cost, standard-gauge.
Grade	$4 200	$272	$4 472
Bridges, trestles and culverts	2 600	168	2 768
Tracklaying and surfacing	1 600	97	1 697
Ties	5 100	737	5 837
Total for these four items	$13 500	$1 274	$14 774

This shows an estimated additional cost of standard- over meter-gauge for a railway of light earthwork and bridging of $1 274, as compared with the author's estimate of $1 560, neglecting general expenses.

The question of gauge, except perhaps in Argentina and Southern Brazil, is not likely to arise on railways of medium or heavy earthwork and bridging, but it is more likely to arise on those railways built in districts of light traffic, where it is necessary to cut down the expense of construction to the lowest possible limit. As stated before, the speaker prefers to do this by building a railway of the gauge standard

Mr. Henry. to the district, high maximum grades and curvatures, say, 3½ or 4% and 25° curves, intelligent discrimination being used in the application of these maxima. By using the Shay or a similar type of locomotive, the maximum grade may be run up to 8 per cent. As one train each way per day—sometimes only two or three trains per week—will take care of all the traffic, it is evident that the cost of conducting transportation is a small item when compared with the cost of maintenance of way and with the interest on the cost of construction. Therefore, under such conditions of light initial traffic and slow development of traffic, the first cost of construction, with due regard to maintenance of way, should be the controlling factor. At first sight, this would indicate a narrow-gauge railway, which admittedly can be built at from $1 000 to $1 500 per mile lower cost than standard-gauge, the interest on which at 7% is from $70 to $105 per mile per year, quite an item for a railway the gross earnings of which may run from $1 000 to $1 500 per mile per year. Still, other considerations must be taken into account, such as gauge of connecting railways or of railways likely to be constructed. In Bolivia the gauge of the railway from Viacha to Oruro, 127 miles, was fixed at 1 m., as it was intended eventually to connect it with the Argentine Railway on the frontier, 425 miles distant, which is of meter-gauge, thus furnishing a through route from La Paz, the capital of Bolivia, to Buenos Aires. In Haiti and Santo Domingo, where the largest mileage of railways is of 42-in. gauge, that gauge will naturally be adopted for future railways in that island. In a country, however, where standard-gauge now prevails, or is likely to prevail, the building of a narrow-gauge railway is of doubtful expediency, and the speaker would prefer reducing the first cost by high maximum grade and curvature.

Mr. Hill. JAMES J. HILL, F. AM. SOC. C. E. (by letter).—In the development of much of the territory of the United States, the original cost of construction of the 4 ft. 8½-in. gauge was all that (and in many cases more than) the country could bear, though the writer has long felt that it is not certain but that, for very heavy traffic, a 5 ft. 6-in. gauge would have been better. The difference in the dead weight of the car would not be great, and the center of gravity would be nearer the rail.

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DISCUSSION ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.*

By Clifford Richardson, M. Am. Soc. C. E.

Clifford Richardson, M. Am. Soc. C. E. (by letter).—This report rings the changes on a number of conclusions with which most experienced road builders are acquainted and with which they will agree. They can, probably, not be brought too often to the attention of those who are less well informed. In some directions, however, these conclusions will not be agreed to by all road builders of experience, nor will there be complete unanimity, as far as approval of some of the definitions proposed by the Committee is concerned. Mr. Richardson.

In discussing the mixing method the report states "Excessive sizes * * *, of the mineral particles should be avoided". It seems to the writer that this statement should be modified. The use of large-size stone, 2½ in., is not objectionable if the road crust is of sufficient thickness to permit it, and if the surface is well closed up with finer material, but it would be absurd to use such a stone in a 3-in. thickness, for instance. The size of the stone should be accommodated to the thickness of the road crust, but where large stone can be used, the surface shows greater stability with large-size particles than with small ones.

Under the same subject, the report says that "Mixing machines should be used, and hand-mixing methods should be avoided wherever practicable". The difficulty with the mixing process lies in the fact that most mechanical mixers will not handle large stone in a satisfactory manner, and that hand-mixing is necessary for such a purpose. In the writer's experience, some of the best roads with which he is acquainted have been constructed by hand-mixing processes, as, for

* Continued from February, 1914, *Proceedings*.

Mr. Richardson.

example, Western Avenue, in Gloucester, Mass., which was built by this process in 1909.

All road builders of experience will agree with the Committee's conclusion that the use of fine sand on top of bituminous concrete is open to grave objection, and that clean stone chips, or small gravel free from particles that will pass a ½-in. mesh screen, is preferable.

Among the Committee's conclusions in regard to the construction of road surfaces by the penetration method is one which states that "An important factor for successful results is the thorough compaction by rolling of the road metal before the spreading of the bituminous material". In the writer's experience, though this may be true where the bituminous binder is one which melts to a slightly viscous liquid and, in consequence, might not remain in place with the stone not thoroughly rolled, it is far from being so when the binding material consists of the more viscous and stable natural asphalts. Experience has shown that better results are obtained in using the natural asphalts for penetration purposes if the rock is merely spread, the road asphalt being applied thereto without rolling, followed by a course of smaller stone, and compaction after this application. The Committee is certainly right in stating that "the use of fine sand on top of the bituminous material is open to grave objections" and that clean stone chips are preferable. No one who has had extended experience in road construction could differ from these conclusions.

Not all engineers will agree with the Committee's suggested definition for an "aggregate", in so far as the statement goes that "the broken stone may be referred to as the aggregate and the sand, stone dust, or other fine material may be referred to as the binder". If such a definition were accepted a sheet-asphalt surface would contain no mineral aggregate.

The definition proposed by the Committee for bitumen is that adopted by the American Society for Testing Materials, as suggested by a Committee of that Society. At the meetings of that Committee and, before the Society, the writer raised very strong objections to the use of the word, bitumen, in a sense other than that of its etymological origin, and presented a Minority Report,* which, as the definition has again been proposed for presentation to the American Society of Civil Engineers, should, he thinks, be brought to the attention of the membership. It is, therefore, here reproduced:

"In the writer's opinion, the definitions presented are unsatisfactory, both in matter and form, and in one instance is quite contrary to the etymological significance of the word defined. The majority report defines bitumen as a 'mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, etc.' The Committee throws over all authorities that have heretofore ruled on the subject, and provides

* *Proceedings*, Am. Soc. for Testing Materials, Vol. XII, p. 75.

that by its arbitrary decision coal tar is a bitumen. It is stated by the Committee that it believes the definitions of the dictionaries and encyclopædias are unsatisfactory, and brushes aside the following definitions of our best known dictionaries and writers: Mr. Richardson.

"'Bitumen: The name given by Latin writers, especially by Pliny, to various forms of hydrocarbons now included under the names asphaltum, maltha and petroleum'. "Century Dictionary, Edw. S. Dana, Ph. D.

"'In modern scientific use, the generic name of certain mineral inflammable substances, native hydrocarbons more or less oxygenated, liquid, semi-liquid, and solid, including naphtha, petroleum, asphalt, etc.'
"New English Dictionary, Murray.

"'Any native mixture of hydrocarbons, oxygenated, as naphtha and especially asphalt'. "Standard Dictionary.

"'By extension, any one of the natural hydrocarbons, including the hard, solid, brittle varieties called asphalt, the semi-solid maltha and mineral tars, the oily petroleums, and even the light, volatile naphthas.'
"Webster's Dictionary.

"'This term includes a considerable number of inflammable mineral substances consisting mainly of hydrocarbons. They are of various consistence, from thin fluid to solid, but the solid bitumens are for the most part liquefiable at a moderate heat. The purest kind of fluid bitumen called naphtha, or rock oil, is a colorless liquid of specific gravity 0.7 to 0.84 and with a bituminous odor. It often occurs in nature with asphalt and other solid bitumens. Petroleum is a dark-colored fluid variety containing much naphtha. Maltha or mineral tar is a more viscid variety. The solid bitumens are asphalt (q. v.) mineral tallow or hatchetin; elastic bitumen, mineral caoutchouc or elaterite; ozokerite.' "Dictionary of Applied Chemistry, Thorpe.

"'A generic name for a variety of substances found in the earth, or exuding from it upon the surface, in the form of springs. The liquid varieties become inspissated by exposure and eventually harden into the solid form, which is asphaltum.'
"Appleton's American Cyclopædia.

"'Bitumen is the name used to denote a group of mineral substances, composed of different hydrocarbons, found widely diffused throughout the world in a variety of forms which grade from thin volatile liquids, to thick semi-fluids and solids, sometimes in a free or pure state, but more frequently intermixed with or saturating different kinds of inorganic matter.'
"From 'Highway Construction', by Austin T. Byrne.

"'The word bitumen may, therefore, be strictly defined as a general term that is used to designate a class of minerals as they occur in nature, that are soluble in chloroform and other neutral liquids.' "S. F. Peckham.

"'Any mixture of hydrocarbons and their derivatives of mineral occurrence whether solid, liquid or gaseous, which is soluble in chloroform or similar solvents.' "George W. Tillson.

Mr. Richardson.

"It seems to me that this is a most arbitrary procedure. If the definition in Thorpe's Dictionary of Applied Chemistry cannot be accepted as a satisfactory one for bitumen, the entire structure of chemical nomenclature might be overthrown.

"As a substitute for the definition proposed by the majority of the Committee, I would suggest the following:

"Bitumen is a material found in nature, consisting of a mixture of hydrocarbons and their derivatives, which may be a gas, liquid, a viscous liquid or maltha or a solid but, if solid, melting more or less readily on the application of heat and soluble in carbon disulphide or similar solvents.

"The definition of 'bituminous' is equally unsatisfactory from the writer's point of view, being defective in so far as it is based on the previous definition of bitumen. The writer would suggest the following:

"A material is said to be bituminous when it contains bitumen or material resembling bitumen, or if it yields bituminous material, or if it constitutes the source of bituminous material. Coal-tar is called a bituminous material from its resemblance physically to some of the denser forms of the native bitumen, although it contains no bitumen. Bituminous macadam is a road surface bound with bitumen or bituminous material."

To this Minority Report, four members of the Committee replied that the application of the term bitumen to coal-tar is sanctioned by Allen in his "Organic Analysis", Dr. David T. Day of the U. S. Geological Survey, Prévost Hubbard, Assoc. Am. Soc. C. E., of the Committee, in his publications, by the American Railway Engineering Association, in the "American Civil Engineers' Pocketbook," and in the Progress Report* of the Special Committee on Bituminous Materials for Road Construction, of the American Society of Civil Engineers, and that bitumen is used, in the sense adopted by the Society, in the specifications of such States as New York, Pennsylvania, Illinois, New Jersey, etc., and by many city authorities like George W. Tillson, M. Am. Soc. C. E.

On the other hand, such an authority as M. Paul le Gavrian, Assistant Secretary, International Association of Permanent Road Congresses in Paris, has stated in reply to an inquiry from the writer:

"I am of your opinion as regards the abuse of the word 'bitumen'. It should be used only to denote natural products and not the tars and their derivatives. I go even farther than you and wish that the term 'bitumen' might be reserved solely for materials containing natural bitumen."

Again, Colonel Crompton, Consulting Engineer of the Road Board of Great Britain, writes:

"You are right in supposing that I agree with you that coal-tar is certainly not bitumen."

* Presented to the Annual Meeting of the Society, on January 17th, 1912.

Mr. Richardson.

The subject is to be considered by the Engineering Standards Committee of Great Britain in conjunction with some of the American authorities, in consequence of a meeting held in London at the time of the recent International Road Congress. In view of what M. le Gavrian and Colonel Crompton think, it hardly seems to the writer that there can be any unanimous acceptance of the American definition for bitumen at the present time, and, this being the case, it would seem to be wise not to attempt, in view of the controversial nature of the matter, to force it on the engineering world, by a majority vote, which will probably chiefly be cast by individuals not directly interested in the subject, even if the majority is very large.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

BY MESSRS. F. LAVIS, H. C. PHILLIPS, WILLIAM W. CREHORE, M. R. MALTBIE, HENRY FLOY, V. K. HENDRICKS, WILLIAM J. WILGUS, RICHARD T. DANA, R. S. MCCORMICK, RICHARD J. MCCARTY, A. W. BUEL, J. E. WILLOUGHBY, S. S. ROBERTS, C. P. HOWARD, JAMES D. MORTIMER, CHARLES S. CHURCHILL, M. H. BRINKLEY, H. M. STONE, S. WHINERY, PHILIP W. HENRY, CHARLES HANSEL, AND D. W. LUM,

Mr. Lavis.

F. LAVIS, M. AM. SOC. C. E.—This report is entitled "Report of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property," etc., with the sub-title "Valuation for the Purpose of Rate-Making." The speaker agrees that it is an important contribution to the literature of valuation; he thinks it unfortunate, however, that, at least by inference, it fosters the belief that the principles formulated and proposed can or may be used in the valuation of railways for the purposes of rate-making.

The title and designation of the Committee, as well as the proposal for its appointment, leave no doubt that the valuation of the railways was uppermost in the minds of the Board of Direction when the Committee was appointed; it seems unfortunate, therefore, that only one railroad engineer was appointed on it.

In view of the nature of the report, the speaker is strongly of the opinion that it is unfortunate that the word railway or any reference to the railways or their rates was permitted to appear in connection with it. He suggests that either the question as affecting railways be entirely eliminated, and a clear statement made at the beginning of the report that such is the case, or that the Committee be increased by

* Continued from March, 1914, *Proceedings*.

Mr. Lavis.

the addition of three or more members of wide experience and training in railroad location, construction, maintenance, and operation. Such contribution as the speaker offers in the discussion which follows, is directed more particularly to pointing out what he considers to be unwarranted inferences in regard to the valuation of railways which might be drawn from the report.

It is noted that in the report the expressions "physical valuation of property" and "valuation of physical property" are used quite indiscriminately. One of the fundamentals of the establishment of principles is the definition of terms. It is suggested that the Committee establish the use of the expression "valuation of physical property" to mean the establishment of the value of the inventoried items, without consideration of the so-called intangible values; also, that in speaking of the determination of the value of the property as a whole, that is, as a going concern, the word "valuation" or "appraisal" be used alone, without the addition of the word "physical." As an instance of the confusion of terms, it may be noted that the Committee includes "the cost of development" in what it calls the "physical value." This cost, of course, is part of the value of the property, but is hardly of the same class as the value of the rails, ties, etc., which are physical. The statement is made that physical valuation "does not furnish a satisfactory basis for rate regulation", but that other things must be considered. What is meant, presumably, is that other things than the value of the physical property must be considered. Among these items are mentioned location, design, and efficiency of operation, but it is admitted that the formulation of principles whereby values of these items may be determined for the purpose of rate regulation is somewhat difficult. Mr. Gibson in his discussion* also advocates the inclusion of such intangible items as strategic location, efficiency, etc.

The speaker has pointed out, in his discussion of the paper† by W. J. Wilgus, M. Am. Soc. C. E., the fallacy of the supposition that rate regulation of any individual railroad could be based on the result of any valuation of any kind; and his point of view and its reasonableness have been specifically and in detail publicly confirmed since then by the Hon. Charles A. Prouty, of the Interstate Commerce Commission.‡ It is out of the question, of course, to assume that the efficiency of the management of, say, one of the four through lines between New York and Chicago could affect its rates, or that its physical characteristics, as shown by its profile, alignment, or length, could do so. These things are items affecting the values of the different lines, but not their values for purposes of rate-making, or even rate regulation.

* *Proceedings*, Am. Soc. C. E., for February, 1914.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 283.

‡ An address before the U. S. Chamber of Commerce at Washington, D. C., February 11th, 1914, *Railway Age Gazette*, February 13th, 1914.

Mr. Lavis.

A most excellent exposition of the various factors governing the establishment of the rates on railroads has recently been contributed by Mr. G. C. Hand,* the details of which it is not necessary to repeat, suffice it to say that he shows clearly that railroad rates in the United States are based on such a variety of different factors, including competition of different lines between the same points, a limited maximum which can be charged to move certain business, the accessibility of markets, balance of traffic, etc., etc., that neither the value nor the cost of service, nor the value of the property involved, are more than one of several factors—and often subordinate ones—in the determination of the rates on any individual railroad.

The Committee has quite wisely taken the position that valuation—of railroads or other property, as a whole and as a going concern—may, and should, be governed by the purposes for which it is made, and realizing the vastness of the subject decided to limit the scope of its inquiry. The speaker ventures to suggest, however, that the subject might well have been, and still might be, narrowed down to the consideration of the question as affecting the determination of the values of the physical property of railroads, or that the railroads be left to their own resources in working out their own salvation.

It is perhaps pertinent to the discussion to point out that though the valuation of the physical property of the railroads is very largely an engineering work, the valuation of the whole property of our railroads can only be properly made by a combination of financial, legal, and operating experience, combined with engineering; and that any committee of this Society would do well to hesitate before formulating principles for the determination of values other than of the physical property.

The Committee has devoted 18 pages of its report to the subject of depreciation, and this has been followed by a further supplementary statement of 18 pages on the same subject, and certain tables have been presented in the addition to the report. Appreciation is disposed of in 2 pages, and its discussion is practically confined to the consideration of increase in land values. The speaker, in his discussion of Mr. Alvord's paper,† expressed the belief that the physical property of any of our first-class trunk-line railways is worth to-day as much as, if not more than, its cost of reproduction new. Mr. Wilgus has expressed a similar belief in his discussion of this report, and it is undoubtedly held by every railroad man of experience, yet we are asked to believe, not only that valuations showing the present fair value of our railroads as only 85% or 90% of the cost of reproduction new are correct and just, but that rates should be based on these values.

The speaker believes that depreciation must be deducted in order

* *Railway Age Gazette*, February 27th, 1914.

† *Proceedings*, Am. Soc. C. E., for January, 1914, p. 191.

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to obtain the true value of the physical property, but he also urges a more careful consideration of appreciation, and allowance for omissions in the inventory, which he believes will show the final value of the physical property alone of an established and well-maintained railroad to be at least 100% of its cost of reproduction new.

As regards the railroads which are not efficiently maintained, a decrease of rates on account of decreased value of the property will not build them up.

This, however, is aside from the main contention of the Committee's report, which deals with the methods of determining an annual charge as a proper allowance for rate-making. So far as the railways are concerned, a much more correct expression would have been renewal charge, or allowance for renewals.

The very clear and illuminating statement* of Mr. Jared How, as to the contractual obligation of the railroads to keep their property at 100% efficiency, and the difference between the continuity of railway maintenance, as contrasted with the life of a pumping plant, for instance, is of much interest.

The determination of an annual charge for renewals on a railroad seems to the speaker to be taken care of quite satisfactorily by an annual charge of a certain percentage of the value of the property based on its estimated life, and there seems to be no use in introducing or attempting to introduce great refinements in the estimate of what this amount should be. The intricate calculation of sinking funds, compound interest, etc., together with an elaborate adjustment of the value to be allowed for the actual depreciation, the latter varying constantly with the assumed fair return and assumed life, does not seem applicable to railroads.

The property of a railroad must be considered as starting with a value of 100, reaching a certain normal state, and remaining constantly in that condition, being maintained there by yearly appropriations to cover maintenance and renewals. Theoretically, these appropriations would be nearly the same each year, but it is easily understood that they may well fluctuate slightly, and it is desirable that they should be able to do so.

There is really no question of compound interest, or sinking fund, or anything of the kind; there is a continuous yearly appropriation to keep the property up to standard efficiency.

The theory of the Committee, however, is based on property which starts at 100, decreases to zero, returns to 100, and so on, and where it is necessary to have set aside—no matter how the money may be kept or invested—a sum for this complete renewal or replacement. At first thought, it would seem as if this was true of any property, but in its application to a railroad, it seems to the speaker that this question of

* *Proceedings*, Am. Soc. C. E., for February, 1914.

depreciation is so mixed up with and related to maintenance that it can only be taken care of or included in and with maintenance expenses, or virtually as an operating cost, and this is where the railroad situation differs materially from that of other types of public utilities. For instance, the cost of renewal of rails and ties is closely correlated to and dependent on not only the quantity and kind of ballast in use, the quantity of track work performed, the climatic conditions, weather, etc., but also on the type and condition of the rolling stock and the amount of traffic. Mr. Lavis.

It may seem to be theoretically possible to take a structure like a large bridge, a great terminal, or similar large item, and calculate a sum to be set aside yearly to replace it, but really the very uncertainty of the length of their life makes this utterly impractical in application.

In a general way, it is known approximately what percentage of the total value of the physical property of railroads should be set aside yearly for maintenance and renewals, and, for all practical purposes, it seems to the speaker that this percentage, based on the experience of many years, is as near as we can come to fixing a fair value for this allowance.

If it is necessary to go into further detail, each item composing the railroad may be tabulated, its life approximated, a value fixed by deducting the scrap value from the cost new, and the yearly charge determined by dividing this value by the estimated life. The bases of these calculations, however, as will be readily seen, are so indefinite and subject so much to judgment and individual experience, that greater refinement than this seems useless, and, as in the case of railroads, this money is actually—no matter what the theoretical case may be—used up each year, it is no use to establish elaborate methods of providing for compound interest, etc.

The Committee has advanced the *reductio ad absurdum* argument, that certain methods of estimating this percentage would mean that the property owners would have, at the end of, say, 10 years, $250 worth of property for every $100 invested. As a practical proposition, however, there seems to be little danger of this, so far as the railroads are concerned.

There is little need to fear, as Mr. Bates* seems to, that fair and adequate regulation of the railroads by the Government will result in the spoliation of the widows and orphans. In spite of the general efficiency of the management of our railroads and the integrity of their officials as a whole, there are not lacking examples of some neglect by them of the interests of the large body of stockholders whose holdings are so small as to deny them effective representation. The speaker has had some experience with regulation in other countries, and he is in-

* *Proceedings*, Am. Soc. C. E., for February, 1914.

Mr. Lavis. clined to believe that the concentration of the regulation of the railways in one national body composed of men not less just and fair than the best in the country, will be a benefit rather than a misfortune, and in this he believes most of the railway officials agree. It is really a national misfortune, that adequate regulation by the National Government was not undertaken from the beginning.

The speaker believes that the valuation of the railroads which has been undertaken by the Government, if it be properly made—as with the co-operation which the railroads are giving, it undoubtedly will be—will constitute a long step forward in the acquisition of that definite, full, and accurate knowledge which is necessary for a clear understanding of the situation, which knowledge is not now possessed by any one, neither the Government, the general public, the stockholders, nor even many of the officials and directors of the railways themselves.

How this knowledge is to be applied to regulation is not clearly seen by any one just now, but the study and the knowledge obtained will undoubtedly help to a clearer view. It seems that it can only be applied to rate regulation in the most general way, and, therefore, as if the work of the Committee might well be directed to the determination of the principles governing the valuation of the physical property alone, this inventory and valuation to be made in such a way that it may be used as a basis for any of the purposes to which valuation may be applied.

Mr. Phillips. H. C. PHILLIPS, M. AM. SOC. C. E.—The speaker regards it as exceedingly unfortunate that the Special Committee assigned to prepare a report formulating the "principles and methods for the valuation of railroad property and other public utilities", after more than a year of silence, and with no inkling of the change of direction of its labors at the Annual Meeting of the Society on January 15th, 1913, should have so limited the subject as to bring into it conditions which, as far as railways are concerned, tend to weaken the whole discussion.

With regard to the report as applied to public utilities which by their nature are somewhat in the class of protected monopolies, or, at any rate, have a well-defined sphere of action, the speaker has no criticism to offer. However, the Committee on the title-page of its report indicates that its subject is "Valuation for the Purpose of Rate-Making", and, further, that railroad properties are included throughout its report. With the exception of trolley lines, rather free from competition and more or less local in their nature, the speaker, in a study of valuation subjects extending over the past two years, has been unable to find any place where valuation has been used in the determination of rates on railroad properties, and though it is entirely true that valuation has been held by the Courts to be the test as against confiscation by rates made on altogether different principles, the

rates of a railroad have been, and of necessity must be, based on many other factors, and valuation carries a very small and insignificant part in the determination of such rates. Mr. Phillips.

To members of the Society having had railroad experience, it is hardly necessary to point out that the factor of competition as between railroads, places, and commodities, and, to a lesser extent, the cost of performing the service, have been and will be the leading factors in the determination of railway rates, and the only point at which valuation becomes a live issue in connection therewith is to determine whether rates made as outlined, or on other principles, are or are not confiscatory, and the fact that in one of the leading rate cases of recent times, where valuation has had its day in Court, the conclusion was finally reached that one of the three roads under consideration would be justified by its valuation in charging higher rates than the other two, if it could obtain them, which, of course, as a practical matter, it cannot.

With this in mind, it seems as if the Society as a body would be exceedingly ill-advised to urge as the result of its deliberations a conclusion that railroad rates are or can be based on valuation, and the report, therefore, should be confined to public service properties (on consideration of which it is mainly based), which by their very nature are limited and protected in their field of activity, and no attempt should be made to cover rate-making for railroads which is based on far different considerations.

WILLIAM W. CREHORE,* M. AM. SOC. C. E.—Broadly speaking, the subject adopted by the Committee for its report, namely, Valuation for the Purpose of Rate-Making, the subjects of certain papers which have been presented to the Society recently, namely, The Appraisal of Public Service Properties, The Physical Valuation of Railroads, The Depreciation of Public Utility Properties, and similar topics, all involve the one important question: How shall a fair basis be determined for the rates to be charged by public utility corporations? Mr. Crehore.

The Real Question.—First, what is the true basis for reasonable rates, and, second, on that basis, what are reasonable rates? When these two questions have been answered satisfactorily, the object of the present discussion will have been attained. The different phases of this subject are introduced to lead by different routes to different conclusions, but these conclusions must furnish an answer of some kind to the question: What are reasonable rates?

The point has been raised in this discussion that a definition of the term "value" must be adopted before it can be used intelligently, that the Courts have defined it in various ways, and that many of those taking part in this discussion have defined it at variance with the Courts. The speaker strongly recommends that some standard

* Previous discussion by Mr. Crehore appeared in *Proceedings* for February, 1914, on p. 380.

Mr. Crehore.

definition be fixed for the term, "valuation", which will be the understood meaning of the term, applicable to all purposes for which a valuation is to be used. To this it will be objected that years of custom and Court decisions have established a precedent requiring property to be valued for taxation in a different way from that required for rate-making or for acquisition. So far as we are bound, by law and Court decisions, to follow certain courses of action, there can be no discussion as to our duty, but being thus bound should not prevent our careful consideration of the propriety of existing laws and customs, or an attempt to change such laws and customs whenever it can be shown that a change would be beneficial. To set aside this whole question with the statement that the conclusions of one's opponents do not conform with existing laws and customs is not argument. The expediency of asking for a change of prevailing law involves a discussion of the effects which would follow such a change, and it is certain that no change for the better will ever be adopted unless convincing arguments are brought forward to uphold its expected advantages, and necessarily such a discussion must precede the campaign of education required to bring about the change.

The Term "Physical Value".—To approach this question with an open mind it should not be taken for granted that a proper basis for rates need be valuation of any sort whatever. The real valuation may be found to be something which is not at all suitable for a rate basis. Entirely apart from the question of valuation, and after its determination, it should then be decided whether or not the true valuation is a proper basis for rates.

It seems to have been pretty well agreed by the majority in this discussion that the intrinsic or tangible value of any property is not its "Sales Value," nor its "Original-Cost-to-Date Value", nor its "Cost-to-Reproduce-New Value", but that it is its "Cost-to-Reproduce-Less-Depreciation Value". The term "intrinsic value" denotes the inherent value, the value residing in the property itself, the tangible value, as distinguished from extrinsic or intangible values, such as good will and other elements of value attaching to the property by virtue of its association with other property, or for some other reason, but not values inherent in the property itself. It will be noted in passing that, under this definition, depreciation could not logically be included in the intrinsic or tangible value of property.

The preceding general discussion has been characterized by much indefiniteness on the point of what constitutes the physical value. Some have attempted to make the term cover intangible values besides the intrinsic value, and others have restricted its use to the intrinsic value alone. Still others have included within its scope all the intrinsic value and a part only of the extrinsic values, expressly excluding certain intangible values. For the most part, it seems to be the general intent

Mr. Crehore.

to restrict the term "physical value" to the intrinsic or the actual tangible value of the property.

The sales value usually includes something for extrinsic or intangible values, and yet, depending on the prosperity of the business or the value of the property due to its association with or separation from other properties, the sales value may either exceed or fall short of the intrinsic value of the property. In the majority of cases, the selling value constitutes a contract agreement between the two parties interested, and their individual circumstances at the time of the sale will sway the price one way or the other in spite of any estimate that can be put on such property. The selling price, however, is influenced most of all by the earning capacity of the property; and, as has been pointed out several times, as we are seeking a basis for regulating the earning capacity, it would be reasoning around in a circle to offer to determine that basis by the very element we wish to make dependent on it.

The merchandise in every modern store is inventoried by the Cost-to-Reproduce-Less-Depreciation Method once a year or oftener, and yet no one would think of buying out a prosperous merchant simply by paying him for the physical value of the goods on his shelves. There must be a further payment for certain extrinsic values, including the good will of the business. The feature of good will and other extrinsic values, in the case of a highly prosperous monopoly, may constitute by far the largest asset, the intrinsic or physical value of its real property being immaterial by comparison. What is it, then, that gives any extrinsic or intangible value to property? It is the earning capacity tempered by the risk involved in its ownership. When a man buys stock, all he wants to know is how much income he can expect from it, and the reputation of the corporation. Satisfactory answers to these questions do not need to be supplemented by information as to the amount of capitalization, the value of the assets, etc., etc. The corporation may not have any assets in sight except the good will of the business, and yet the stock may be a first-class investment.

Physical or Intrinsic Value Inadequate as a Basis for Rates.—It is plain that the physical value, that is, the intrinsic value of property of all kinds, can be estimated with a fair degree of accuracy, but that the extrinsic or intangible value depends for its size directly on the earning capacity of the unified business taken as a whole. As we cannot seek a basis for regulating the earning capacity in something that is dependent on that earning capacity for its very existence, there is no part of the value available as a basis for rates except the bare physical or intrinsic value of the property. As already stated, it is not necessary that valuation of any sort be made the basis for rates, but it is apparent that if rates are based on the bare physical valuation of the property, without any allowance for return on certain intangible values, the return on the investment will not be adequate.

Mr. Crehore.

Many items of intangible value have cost the investor large sums of money. Articles of trade often have to be given away to introduce them to prospective customers. Advertising is very costly, and some business enterprises require two or three years of advertising before they can show a profit on their sales account. There is nothing to show for these and other large initial expenditures except intangible values, generally designated as the good will of the business. Yet, in some lines of business, the good will is the chief asset, because the actual amount of tangible property required to keep them going is so very small. The professional man with a large and lucrative practice would certainly not care to sell out for the value of his tools or other physical assets required to keep up his business.

A toll bridge company may find at some stage of its existence that the physical value of its bridge and other property less depreciation is $20 000, and yet may be able to show annual profits which, if capitalized at their present worth and treated as an annuity, would require a fund of $30 000 from which the equivalent of the present actual profits of the company could be withdrawn annually so that the fund would just be exhausted at the expiration of the life of the company's charter, the probable life of the bridge and the company's charter being coterminous. Such a fund would then be the proper measure of the total value of the property, including the intangible as well as the tangible.

Although the examples just cited may not be strictly applicable to the case of a public utility, it is well known that there are many hundreds of items which enter into the cost of launching any large enterprise, whether it be public or private, and some of these items never produce any beneficial effect on the business, while others more than make up for them and give it a very material intangible value, which is just as real and costly as though it were possible to see, feel, and estimate its items.

It is apparent that the earning capacity of any property is the vital point in the determination of its total value, both extrinsic and intrinsic; yet the intrinsic value only can be determined by the Cost-to-Reproduce-Less-Depreciation Method. As intangible values are indeterminate, except by means of the earning capacity, which is what we are seeking to fix, it therefore seems reasonable to conclude that a just and equitable value on which to base rates cannot be found. It must be assigned, or approximated, or guessed at. In truth, the problem of evaluating public utilities is the reverse of the ordinary problem met with in dealing with private corporations, and begins with a determination of the minimum return that can be offered to attract capital. A private corporation, on the other hand, is in business to make all the money it can for its stockholders, and is supposed to be regulated by the power of competition.

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The Fair Allowance as a Basis for Rates in Excess of the Physical Value.—Mr. Grunsky, in his paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates", said:

"Intangible values, of whatsoever nature, result from high earnings. In the case of public service corporations, they are arbitrarily created by agreeing to, and permitting, rates which produce a revenue in excess of the ordinary return on safe investments. They do not exist unless the rates are higher than those which would produce net earnings equalling an ordinary interest return on the properly invested capital."

An "ordinary interest return on the properly invested capital" is elsewhere spoken of by the same author as "the return on ordinary safe investments" like savings bank deposits or other investments of like character. Mr. Grunsky, in course of his able presentation of the subject, takes the ground that intangible values should be disregarded in making appraisals for rate-fixing purposes, but that the net earnings of a public service property should in some measure exceed the return from ordinary safe investments. They should, indeed. It is certain that capital could not be attracted at all to our public utilities unless there was a prospect of greater return than is offered by savings banks or Government bonds. As just stated, the public utility problem begins with a determination of the minimum return that can be offered to attract capital. In explanation of his position in this matter, Mr. Grunsky says:

"reference may again be had to the case of an owner of a public service property who invests only borrowed money. If he receives only such interest on the investment as he must pay to the bank, he will have rendered a service without compensation, except such as he may be allowed in salaries, under operating expenses. In such case, it would be a proper business arrangement to compensate him for the risk of loss which he assumes, and for his management, and to make this compensation in some measure proportional to the net earnings. If the owner in the cited case is a stock company, this compensation will be the only element giving value to the capital stock of the company".

As one method of fixing a basis for rates, when finally the physical or tangible value itself has been found, it will be possible to make rates on it so large that the return to the investor will include something for certain assumed intangible values, that is, the percentage of return may be made any desirable degree larger than the return on ordinary safe investments. Perhaps this is the method which would be adopted by those who contend that the physical valuation alone should be made the basis for rates. The trouble with this is that to allot a fixed percentage of the physical value as the return on the intangible values is to assume that these two values always bear the same ratio to each other, which is very far from true. Certain it is that, without some

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allowance in the rates to remunerate intangible values, there would be little attraction for investors. The necessity for adding something to the purely physical valuation when used as a basis for rates is apparent; but, as to what that something should be, there has hitherto been no agreement. Although the different classes of public utilities would require different treatment in this regard, the speaker believes that some average basis could be worked out for each class by itself that would provide an adequate remuneration for those intangible values which originally caused the investors some material outlay.

The Investment as a Basis for Rate-Making.—One of the methods for fixing a basis for rates, seemingly overlooked in this discussion, is to determine the investment. This is not at all what is meant by the Original-Cost-to-Date Method of valuation. There is a clear distinction between the actual cost of all the property purchased by the corporation and the actual amount of cash or property contributed to its treasury by the investors. If the latter could be ascertained by any practical method of accounting, it would be the ideal basis for rates. There would be substantial justice by this method, for the application of it would work both ways. It would take care of the investors' rights, in the event of mismanagement or unwise expenditures of the corporation's funds, and it would prevent burdening the rate-payers with a load of fictitious capitalization. It would also return to the rate-payers the benefit to be derived from a reinvestment of any portion of the earnings and the unearned increment on land values, removing these valuable sources of appreciation from the investors and giving them to the community, where they rightfully belong.

Choice of Valuation by the Owner.—In view of the many unsatisfactory features surrounding and pertaining to all methods of valuation for rate-making hitherto considered, and in view of the enormous amount of labor and expense required to put any one of them into execution, the speaker again suggests for serious consideration the expediency of taking the owner's valuation of his own property and using that as a basis for rates. There should be an end to all juggling with the term "valuation". There should not be one kind of valuation for rate-making, another for acquisition, and another for taxation purposes. If the owner is made to look at the question from both sides, he is far and away the best expert as to the true value of his own property. The way to make him look at it from both sides is to require him to pay taxes on the same valuation that he is allowed as a basis for his income. Within certain limits, it should be the province of public authority to prevent grossly irregular and inappropriate valuations, but, in general, the owners of public utilities should be required to name their own valuation, with the understanding that it was to be used as a basis for rates as well as for taxation, or for acquisition, or for any other purpose.

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Think of the problems it would eliminate. Who ever heard of a public utility paying taxes on a fictitious valuation of any sort if the opportunity had been given to swear them off? Would any public utility care to pay taxes continuously on the proportion of intangible values which it now claims should be rewarded in the rates? If valuations continued to increase, due to the capitalization of earnings, either past or future, at least the rate-payer would have the satisfaction of taking his share of the earnings on these increases through the tax budget; whereas, if utilities in general neglected or refused to declare these additions to their property for taxation, the rate-payers would stand exactly in the same position as if they owned these increases outright, and would enjoy a corresponding reduction in the rates on that account. In the last analysis, the public would remain in control, both of the tax budget and of the rate-making, so that any consensus of action, by the utilities overbalancing the situation either one way or the other, could be readily corrected at the proper time.

At least, while we are waiting the many years it will take to get a physical valuation of our public utilities, let us try the alternative of letting the owners set their own valuation, with the common understanding that it is to be used for all purposes where required.

There is another point to be considered: After the proposed stupendous task of valuation is finished and the returns are in, how long will it be before conditions will have changed so much as to require another similar valuation? General conditions will certainly call for a readjustment of rates periodically, and how can this be done intelligently on the basis of an out-of-date physical valuation? Is this task, which is already recognized as Herculean, going to be repeated as often as a readjustment of rates is required? Rather let us have the task divided up and discharged by the owners themselves who are intimately familiar with the conditions, and then it can be repeated as often as the convenience of the public and the interests of the utilities demand.

Frequent Rate Regulation Undesirable.—The magnitude of the interests involved renders it extremely undesirable that rates should be changed too frequently. To readjust them once in 5 years, immediately after the utilities had filed their valuations, would probably be often enough. It might be too often. It would be preferable to have the period between adjustments too short than too long, however, because, if not long enough, the opportunity may always be availed of to continue the same rates for another period. Reasonably frequent periods would give the utilities a better opportunity to readjust their valuations with regard to all the elements to be considered.

Excessive Bonding and Capitalization of Earnings.—Recently, the Interstate Commerce Commission called attention to the manipulation by the Chicago, Milwaukee and St. Paul Railroad Company of the accounts of the Puget Sound Railway Company, a subsidiary of the St.

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Paul, saying that it was done for the purpose of misleading the public and creating a market for its new bonds. The Commission claimed that this was unnecessary, as the Puget Sound Company could show reasonable profit on its operations without any manipulation of the books, and was in fact on a sound basis, both financially and physically. The President of the Puget Sound Company forthwith explained to the public that the only manipulation of which the Company might possibly be accused was rendered necessary by an effort to obey the laws of the State of Washington, and, in so doing, the bookkeeping was forced to be a little out of compliance with the mandate of the Interstate Commerce Commission. A significant fact of the President's statement was that the Puget Sound Company's original capitalization had been $3 000 000, but when the construction of the Puget Sound Extension was nearly completed a mortgage for $200 000 000 was found to be necessary, and such a mortgage was thereupon issued. The excuse for the irregular bookkeeping previously mentioned was that, under the laws of the State of Washington, it was impossible to issue a mortgage for more than twice the par value of the capital stock of the Company, and, therefore, the capital stock had to be jumped suddenly from $3 000 000 to $100 000 000. Naturally, it would require an agile management to stretch out the hitherto reasonable and sufficient return on $3 000 000 so that it would look anything like adequate for a return on $100 000 000 in addition to the carrying charges and amortization of a $200 000 000 bond issue.

Now, the phase of this situation which challenges our attention is that any company or individual should have the hardihood to ask for a loan of $200 000 000 on property capitalized at only $3 000 000. The first reply to this is that the property was worth much more than the amount at which it was originally capitalized. Let us assume that it was. Let us assume that, by the reinvestment of earnings since the time of its organization, the property had attained an actual value of $100 000 000, the figure at which it was finally capitalized. Even then, the amount of the proposed mortgage was twice the value of the property, and its issue was specially sanctioned by a law of the State. Is there any private business, corporate or other, the managers of which would ask for and expect to get a loan amounting to twice the amount of the security offered as collateral? Rather is it more reasonable to expect that even the best terms obtainable on a mortgage of this kind would be one-half or 60% of the value of the property, instead of twice its value. Yet there are those who would have us believe that a public utility is on no different footing from that of any private corporation.

It seems that the financial managers of public utilities always rely confidently on a steadily increasing valuation of their property through the reinvestment of earnings. They assume that out of future earn-

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ings they will always be able, not only to cover the current depreciation, but also to carry and amortize bond issues far in excess of the present value of the property, and pay a profit besides. In other words, they use bond issues instead of subscriptions to the capital stock, because in so doing they are getting capital free of charge from the public, and whatever profit they can make over and above the interest and sinking-fund payment on the bonds is so much clear gain without the expenditure of a penny to obtain it. Generally, in years gone by, the steady increase of earnings justified this tendency to rely on the future; but in recent years the rate-payer has interposed his veto against supplying capital as well as income. When the appraiser comes along to ascertain the physical value of such property and finds large accessions to it due to the investment of the proceeds of bond issues, the question naturally arises: Should the valuation of this property be credited to the corporation and be made the basis of rates charged to the public in addition to the carrying charges and amortization of the bond issues? The plain statement of the question indicates the injustice of the process, and yet this was, until very recently, and is even now, when not prohibited by law, one of the ordinary methods of financing a railroad.

The reinvestment of earnings over and over again for many years and the discounting of future earnings by bond issues have increased the value of railroad properties at the expense of the rate-payers all over the country far beyond the stockholder's actual investment, and often beyond the capitalization. The discussion of rate allowances must take into consideration the fact that an enormous percentage of the values now found to exist in public utility properties was contributed by the public in the form of excessive rates, and, instead of using this part of the valuation as a basis for increased rates, it should in justice be used as a basis for the reduction of present rates, so that the actual investors in the property would be adequately remunerated, whether they are the rate-payers or the subscribers to the capital stock.

There are those who say, in reference to this matter: "Oh, let bygones be bygones, but let us legislate for the future so that the public may not be forced to contribute capital as well as income to the owners of our public utilities". In urging this view of the matter, it is argued that the present stockholders would be the actual sufferers, as they have paid the current prices for their holdings, and the current prices were based on the earning power of the properties averaged over a period of years. It is this fact that has halted the Interstate Commerce Commission in its treatment of the question of capitalization of earnings, and it is this phase of the matter which perplexes all who have given attention to an analysis of this subject. To base rates on the present valuation of these properties would be a fair solution of the problem for the present security holders, but would be penalizing the

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rate-payers by compelling them to contribute income to the security holders for that portion of the value of these properties for which the rate-payers themselves furnished the capital. For years the reinvestment of earnings, that is, of capital supplied by the rate-payers, has provided our public utilities with an argument for increasing the rates, whereas, on the other hand, it should be an equally potent argument for reducing the rates, if the public was honestly given the benefit of its investment.

The public's plaint, however, does not end here. It has been shown that often and often bond issues by public utilities have been refunded and carried indefinitely long after the property originally provided by their proceeds is worn out or has become obsolete. No effort has been made to retire these loans with the extinction of the property representing them, but they are refunded again and again, and are claiming their share of the earnings of the corporation for something which has long ceased to exist. If the depreciated or obsolete property has been replaced out of the earnings without an accompanying retirement of the bonds, is it not plain that the property and the bonds cannot both claim their share of the earnings without putting a double load on the rate-payer? Is it not just as clear, also, that the process of rolling up the value of the property year after year, like a big snow-ball, out of the earnings of the corporation, calls for continually increasing rates to provide additional income for the security holder derived from the rate-payer's own investment? Is it not evident that the more the rate-payer helps the corporation along on its capital account, so much the more is he penalized by having to pay rates which will cover an adequate income on the ever-increasing valuation? After some years of this cumulative process, the public is now paying rates which are enormously in excess of a reasonable return on the capitalist's original investment; yet the property value is there, and would appear by whatever means the valuation was undertaken. Furthermore, if this process continues in the future, there is no end to the advancement of rates; the more they advance the more they will have to advance to cover the income on their own advancement. It is automatic.

If the rate-payers are also the owners, then the capitalization of earnings is a perfectly proper procedure, and has the effect of automatically reducing the rates any way; but, if the rate-payers are not the owners, it certainly will not be claimed by any one that they should be called on to furnish all the capital used in the business. Then just how much of this capital is it proper and right that the rate-payers should supply, and how much should be furnished by the stockholders? The answer to this question will be one element of the basis for fixing the rates.

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Water.—So far as the rate-payer is concerned, however, this load of furnishing the capital for public utilities is negligible in comparison with the one he is carrying as a result of fictitious valuation. This feature is one that the present campaign of rate-fixing is designed largely to correct, and the present holder of the securities will have to stand the "squeeze." There is no help for him in this regard, nor should there be. He may have been innocently drawn into the investment without a proper investigation, or he may have relied on the movement of stock quotations in forming his judgment of the value of his intended investment, but in any case he has lost by investing in something of which the value was fictitious, and there is no more reason for protecting him in this matter than for protecting the investor in a private banking business who found that foreclosure on certain loans hitherto believed to be impregnable had developed a large shrinkage. To attempt to save the present stockholder from loss in these cases would be saddling forever on the public an almost intolerable burden of fictitious values.

Considering this question broadly, however, such cases of overvaluation as have resulted from the rascality or deliberate manipulation with intent to deceive on the part of one man or a small coterie of men, might receive special treatment in order that the security holders in these enterprises may not be entirely robbed. It might be found feasible in some of these cases to divert some of the surplus earnings now being reinvested as capital and apply them to a gradual reduction of the holdings of innocent stockholders who were the victims of the rascality of the managers. This, however, comes more properly under the head of financial management, and is outside the topic now under discussion.

Land Values as Excess Earnings.—Even if the percentage of excess earnings taken from the public to be used as capital by the public utilities is small, it makes a large difference to the capitalist in the amount of his actual investment. Constantly adding to his holdings from the capitalization of earnings, the original investor finds his income increasing in times of general prosperity with the acceleration of a compound interest account, without effort or expenditure of his own. References have been made by several of those participating in this discussion to the treatment of land values in connection with public utilities, and Mr. Sparrow has handled this phase of the subject ably and clearly. Why should not the appreciation of land values be treated like values resulting from a capitalization of earnings? The unearned increment in the value of land is no more and no less a gift from the community than is the increase of capital from excess earnings. The added value of public utility property due to reinvestment of a part of the earnings each year has precisely the same effect as the appreciation of its land values, adding to the assets of the investor without

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any expenditure or effort on his part. The only distinction that can be made between the two is that the unearned increment of the land costs the community nothing, whereas all additions to the capital account out of the earnings come directly from the community in the form of high rates. The speaker fails to see any reason why both these questions should not be accorded the same treatment.

It is just as true of any private corporation as it is of a public utility that the reinvestment of a part of the earnings as capital is a free gift from the community, but a private corporation maintains its rates (or is supposed to maintain them), in open competition with all others in the business, whereas a public utility is under contract with the State to perform certain functions in return for which it is accorded a practical monopoly in the district which it serves.

Is the Capitalization of Earnings Confiscation?—Opponents of the proposition to deduct the accrued depreciation from the value of public utility property in establishing the rate-fixing value are very sure that to exclude this depreciation from the rate basis would result in a form of confiscation of the owner's property. This has been loudly proclaimed, and argued with some force. Do those who take this position also find any evidence of confiscation of the rate-payer's property in the capitalization of earnings? If it is taking property from the corporation without due process of law, to exclude from participation in the earnings such capital as has been used up or has become obsolete, or such portions of existing capital as have been allowed to deteriorate and have not been kept up to their full valuation or are without the means to supply this lack, then it is no less confiscation to take from the rate-payer funds to be used as fresh capital on which no part of the return is shared with the rate-payer. The speaker does not think the term "confiscation" applicable to either of these processes, but believes that they are on the same footing with respect to the import of that term.

Efficiency Its Own Reward.—As intimated by the Committee, one of the most important of the intangible values, and one having an amount totally disproportionate to the physical value, even among utilities of the same class, is the item of efficiency. Of two public utilities having practically the same physical valuation, one may be far more efficient that the other in handling its business. A uniform percentage based on the physical value in each case would remunerate each of them alike, taking no account of the superiority of one over the other. It appears to some that such treatment would penalize the more efficient company and put a premium on inefficiency; but, would not the more efficient company find its reward in the resulting lower cost of operation, and would it not, even at the same rate of remuneration as its less efficient rival, by reason of its very efficiency, have a greater margin of profit left for its owners?

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A uniform percentage on the total physical valuation, as the basis for uniform rates, would include an allowance for operating and administrative cost, so that the uniform rates would mean greater profits to the company having the lower cost. Of two utilities covering practically the same territory and serving populations of equivalent size, one might have its capital invested in thoroughly modern equipment of higher value and greater efficiency than that of the other. Appraisals of the two would show for one a larger physical valuation than for the other, and, consequently, the total general valuation for establishing the rates would be higher the more of these modern plants there were. If, as before, the allowance for rates was a fixed percentage which covered the total cost of operating as well as the reward to capital, then the more modern plant, with an equipment having admittedly a higher efficiency than that of the older plant, will reap a further reward for its investors by the lower cost of its operation. In the same way that all other lines of business are compelled by competition at certain times of their existence to discard old and worn out or half worn apparatus for new and modern appliances, just so a public utility finds its own reward in making the change.

It has been suggested in this discussion that if rates are adjusted for the highly efficient competitors in the same territory, the poorer of these will lose until they meet the competition. It is understood, of course, that rates will be based on an average of conditions and valuations representing all utilities in the same territory. On such an assumption the writer agrees with Mr. James that the stronger and richer competitors might be required to pay the State, in the form of taxes, all the excess over the earnings of their weaker rivals, such rebate to the State being arranged on some sliding basis that would preserve the required incentive to efficiency intact; and it would be well to go farther, and limit the time within which the poorer and weaker would be required to put their equipment on a modern efficiency basis or else have the protection thus afforded withdrawn. Rates must be made uniform, whatever system is adopted, and cannot be based on conditions which would suit a single utility or group of utilities in any one class.

Other Problems Concerning Rates.—There are many other problems more important in their results to the public than the problem of finding a rate basis. After the basis is found, rates must be adjusted to it; but, in the process of adjusting them, there are thousands of classifications of which to take account. In the case of railroads, the most complex problem of the whole is that of making the various classifications. These classifications now furnish the basis of nearly all the discrimination prevalent throughout the United States, and it is these classifications and other discriminations, rather than the economic law of supply and demand, that determine in a large number of

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This classification problem is a very complicated one, and this fact has been taken advantage of by the railroads very extensively to assess the public for all the traffic would bear, regardless of uniformity. The Interstate Commerce Commission has done a large amount of work in the past in adjusting these classifications, but still more remains to be done, and there always will be more to do. New questions are continually arising which cannot be foreseen, and, with the railroad companies presenting only one side of the case, it is the duty of the public to present its own side as clearly as possible for the benefit of those in authority who have the final adjudication of these matters.

Mr. Maltbie. M. R. MALTBIE, ESQ.*—There is one thing in the report which has not been mentioned but which seems to be of special value. The Committee has pointed out that the question of the amount upon which a company is entitled to a fair return, whether cost-to-reproduce-new or depreciated value, is inseparably connected with the method of computing the allowance for annual depreciation. Many writers have failed to appreciate the relationship between these two, and have used one method for computing accrued depreciation and another for computing the annual depreciation. The additional discussion by Mr. Stearns has emphasized the point made in the original report, and demonstrates conclusively that one cannot fairly adopt one method for determining accrued depreciation and another for determining annual depreciation. It would be grossly unfair, as shown by Mr. Stearns, to adopt cost-to-reproduce-new as a basis for reaching fair value, and the straight-line method of depreciation in computing the annual allowance.

There are many other factors which the Committee has similarly treated, and indeed, it is probably doubtful whether there are many factors (perhaps there may be a few) where it is possible to settle one question or determine one factor without reference to the determination of others. If the Committee had emphasized and reiterated no other principle, it would deserve our thanks and commendation, for writers and experts have been prone to slip from one thing to another without appreciating the interdependence of the different factors.

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The fundamental question in rate cases is the rate, and the determination of a fair and reasonable amount collected from the consumers or those using the utility. In order to arrive at this, it has been customary to determine the fair value of the property and the rate of return; but, after all, there is no one yardstick which will apply to all cases, and it is only by threshing out the various questions, by clearing the atmosphere, which is often befogged, that it is possible to determine the final outcome; and two plans may bring the same result, but one may differ from the other in practically every point. The report of the Committee indicates that this idea was fully appreciated, and it has attempted with great care and forethought to elaborate a statement of principles which will do justice to the public and to the company.

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HENRY FLOY, M. AM. SOC. C. E.—Before taking up the discussion of the Committee's report, it might be well to call attention to the impracticability of the method of valuing public utility property proposed by Mr. Crehore. This is to let the owner place a value on his property, with the understanding that the rate of return allowed and the taxes to be paid thereon shall each be figured at the same percentage of the value of the property. Although not having any investments in public utility properties, the speaker thinks that all the utilities would be glad to accept any such basis of valuation and pay taxes to the extent of 10, 20, or even a higher percentage thereon. A little reflection will show that, as taxes are paid by the public—the utility merely acting as a collector—the corporations would be perfectly satisfied to pay any amount in taxes, provided the same amount was allowed them for dividends. It will be seen, therefore, that such a method of valuation would be not only impracticable but manifestly unfair to the consumer.

With regard to the suggestion of Mr. Lavis that the membership of the Committee be increased by the addition of three railroad men, the speaker would heartily endorse such procedure, stipulating, however, that the new members be specialists in the matter of appraisals and rate-making—not merely construction or operating engineers. A further suggestion would be that the Committee have added to its membership at least one capable attorney who has made a special study of the decisions of Courts and commissions in relation to the matters covered by the report of the Committee.

An examination of the Committee's report shows the painstaking effort and high character of the work of its members. Credit therefor, and the thanks of the Society, are due the Committee, even though we may not agree with its conclusions.

The title of this report would lead one to expect that its recommendations could be followed in appraising existing utility properties. As a matter of fact, the report deals largely with methods proposed to

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be applied to corporations which have been very recently created or which may be established in the future.

The serious question before engineers at present, is, not what possible theories and methods may be developed for ascertaining the value and determining the depreciation of utilities yet to be created, but rather what methods of valuation shall be followed in appraising the many existing and long-established utilities.

Probably many engineers—and certainly the speaker—would not differ with most of the Committee's report and recommendations, provided such recommendations are applied only to newly created corporations in which investors have placed their money after due notice as to the methods which would be applied and the returns which would be securable therefrom. Few will find fault if a commission, following the Committee's recommendations as to the "tango" treatment of return—a dip at each successive step—can induce capital to invest under such proposed rulings, but it is absolutely unfair and illegal to attempt to saddle untried, theoretical methods on innocent owners, who have invested their money under conditions and upon assumptions entirely different from those now proposed by the Committee.

On this account, much of the report will be of but little help to the Profession in determining proper methods of valuing long-established, existing utilities. To be sure, the report recognizes the necessity of using the so-called "reproduction-cost-new" method of getting at the value of long-existing utilities, but the practical application of such method is only generally indicated.

Passing over the many excellent points in the report that relate to making up an inventory, preliminary, engineering, and development expenses, etc., it would seem as if the Committee was attempting, without sufficient reason, to sweep away many of the fundamentals that have been generally recognized and are now accepted.

The Committee seems to assume that interpretation of the decision of the Supreme Court in the Knoxville Water Company case is the only true gospel. The Committee then proceeds to argue that other leading decisions (more numerous, and as to which the interpretation is less debatable) of the same Court are in error and must be corrected to conform with the Committee's views as to the proper method of making valuations.

As is quite generally known, the Knoxville Water Company case was not adequately prepared, or well presented, in comparison with present-day knowledge of these matters, consequently, the resulting decision and many of its dicta have been erroneously accepted, both from misapprehension of the facts, and misinterpretation of the Court's meaning. If the Committee hopes to attempt the reversal of any Supreme Court decisions, it should attack some of the principles

claimed in the Knoxville decision, rather than those others, which relate to cases that were more adequately prepared, better presented, and more recently decided. Mr. Floy.

In considering any decision of the Supreme Court, it must be borne in mind that the Court is passing on the particular case in question, and any conclusions reached are based on the peculiar circumstances surrounding that particular case. Consequently, too much importance should not be attached to the decision of the Supreme Court in some particular case when attempting to arrive at fundamental principles, which may be applied generally. Although it is true that the last word in any particular instance rests with the Supreme Court, and that such Court is given to following precedent, nevertheless, it cannot be presumed that the Court will seek to perpetuate what may be demonstrated to be a wrong, merely to avoid reversing itself.

No question can be regarded as finally settled, until settled equitably. The reference by Mr. Humphreys to the decision of the Supreme Court, in the Consolidated Gas case, that to maintain a pressure of 2½ in. of water in the gas pipes in a distributing system, "the mains and other pipes would have to be strengthened", is so contrary to fact that eventually the Supreme Court must reverse itself in this matter. In the same way, the statement of Judge Hughes, in the Minnesota rate cases, to the effect that "We also think it was an error to add to the amount taken, as the present value of the lands, the further sums calculated on that value, which are embraced in the items of 'engineering, superintendence, legal expenses', 'contingencies' and 'interest during construction'"; or again, the statement in the same decision, "It is impossible to assume, in making a judicial finding of what it would cost to acquire the property, that the Company would be compelled to pay more than its fair market value. It is equipped with the governmental power of eminent domain", is so contrary to common knowledge and experience in railway construction that the establishment of such principles cannot be accepted, even when attempted by the Supreme Court.

The Committee seems to require some legal support for making deductions from reproduction-cost-new, and, therefore, ties to the Knoxville case, and has developed a unique theory of estimating and providing for depreciation, the allowance therefor being stated to be "in effect, a payment from the rate-payer to the corporation of a part of its investment" (page 32). This is certainly a novel definition, and one that is contrary to accepted practice. The Committee, without apparent warrant, remarks (page 72), with reference to its proposed "Equal-Annual-Payment Method of Determining Depreciation Allowances", that "this method conforms to the decisions of the highest Courts". No references are given as to what Court decisions are

Mr. Floy. considered by the Committee, to justify their proposed method of annually increasing payments to be made by the rate-payer on account of depreciation and decreasing his payments for fair return. The Supreme Court certainly disapproves any such method as that proposed by the Committee, and the speaker suggests that the Committee produce the opinion of a competent attorney to the effect that any Court decision rendered to date is based on any such theory of ascertaining present value as proposed by the Committee's "Equal-Annual-Payment Method".

It is conceded that the decisions of some Courts hold, that, in determining the value of property on which to fix rates of return, deduction from cost of reproduction new, on the basis of a more or less theoretical computation, should be taken into account, but this is not the ruling of the United States Supreme Court. This particular question, with reference to whether the amount of depreciation of physical property was to be determined from actual inspection or from calculations based on life tables, was squarely involved in the Consolidated Gas case and definitely passed on by the Master appointed to take the testimony in that celebrated case. The total valuation of that property was some $56 000 000. It was shown that an expenditure of $604 988 for "repairs" (less than 1%), would make the plant as good as new, and this is the only sum that was deducted from the reproduction cost new, notwithstanding it was most vigorously contended by the plaintiff in that case that a further deduction of millions of dollars should be made for theoretical or accruing depreciation. The Master, in answer to this contention, says:*

"For the purpose of determining present value, however, particularly on the basis of cost of reproduction, the method followed by Mr. Marks [witness for the plaintiff who used theoretical depreciation based on life tables, similar to those proposed by the Committee] does not commend itself. It appears from the record, without substantial dispute, that while certain of the plants and apparatus may not be in perfect repair, they are, as a whole, in efficient operating condition, and that a large proportion of their capacity is represented by the latest pattern of water gas apparatus installed within the last few years."

The Master, in this case, in dealing with the alleged necessity for a reserve fund to provide for "final renewals" when the life of the apparatus should expire, says:

"Of course, the requirement of such 'final renewal' provision affects in no way the present value or efficiency of the plants of the company as operating concerns, except to the extent of the repairs [$604 988 above mentioned], which would be required to make the operating plant as good as new."

* Master's report, p. 110.

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The Circuit Court and the Supreme Court sustained in all respects the valuation fixed by the Master in this case, except on the question of franchises.

It is thus seen from the foregoing decision that property which is in good order and operating efficiently, although not new, need not necessarily be depreciated, at least in rate cases.

It is sometimes contended that the decision in Knoxville *v.* Knoxville Water Co., 212 U. S., 1, decided the same day as the Consolidated Gas case, holds a different doctrine, but a careful reading of that case, in connection with the Master's decision, shows otherwise.

Depreciation which can be determined by "a detailed examination of the property as it stands to-day,"* covers that deterioration due to inadequacy, obsolescence, deferred maintenance, or property not actually used in the service being rendered. It is these various classes of depreciation just specified, and not theoretical calculations based on office records as to date of installation, that are referred to by the Supreme Court in the Knoxville and other cases where the specific statement has been made to the effect that depreciation must be deducted in determining the values of property to be used in fixing rates. In the Knoxville case, the Master stated that he had added "complete and incomplete depreciation" to the present value of the surviving parts, in order to obtain the total value used as his basis for fixing rates. Consequently, the Supreme Court very properly criticized such procedure. It was equivalent to taking the depreciated value of the existing property and adding thereto the cost of all property which has been used in the past and which no longer existed, the value of which under adequate rates, should have been repaid to the investors out of revenue as a part of operating expenses, during the life of the now superseded property.

The Supreme Court says that the Master erred in adding the sum of "complete" and "incomplete" depreciation to the depreciated present value; it does not state that he would have erred in adding only "incomplete" depreciation to the depreciated value. The Court could not have considered that adding the amount of "incomplete" depreciation to the present depreciated value would have been error, otherwise it could not logically have stated that renewals must be paid for out of revenue as part of operating expense, for,

"if a different course were pursued the only method of providing for the replacement of property which has ceased to be useful would be the investment of new capital and the issue of new bonds or stock. This course would lead to a constantly increasing variancy between present value and bond and stock capitalization, a tendency which would inevitably lead to disaster to the stockholder or to the public, or both."

* Master's report, Knoxville case.

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It is evidently the thought of the Supreme Court that present value and the amount of capitalization, as represented by stocks and bonds—that is, the investment value—should correspond, or tend to equal one another, which would not and could not be the case if "present value" were taken to mean present theoretical depreciated value. This view of the Supreme Court is in harmony with its decision in the celebrated Consolidated Gas case, where only observed, actual, evidenced depreciation, obsolescence, or deferred maintenance was deducted.

Nor does the decision of the Supreme Court in the Minnesota rate cases decide that theoretical depreciation should be deducted from reproduction new to arrive at present value as a basis for rates. This decision, so far as it dealt with the questions of valuation, has been quite generally criticized by men best equipped to form an intelligent judgment on such matters. There are a number of propositions in this part of the decision that certainly will not stand.

At page 457 the Court says:

"It is also to be noted that the depreciation in question is not that which has been overcome by repairs and replacements, but is the actual existing depreciation in the plant as compared with the new one. It would seem to be inevitable that in many parts of the plant there should be such depreciation, as, for example, in old structures and equipment remaining on hand. And, when an estimate of value is made on the basis of reproduction new, the extent of existing depreciation should be shown and deducted. * * * And when particular physical items are estimated as worth so much new, if in fact they be depreciated, this amount should be found and allowed for."

If this quotation means anything, it would indicate that the depreciation of the theoretical, estimated kind, which is normally "overcome by repairs and replacements" is not the kind of depreciation being considered by the Court, but only "actual existing depreciation." This is shown, for example, by the deduction for value of "old structures and equipment remaining on hand," after having served their useful life, and held, perhaps, for re-sale as scrap, or retained at full value in its books, by the corporation hoping to be allowed thus to increase artificially the "fair value" of its property on which the rate of return is to be allowed. In the use of reproduction new, "the extent of existing depreciation should be shown and deducted." This repeated use of "existing depreciation," and the word "actual" certainly indicates that class of depreciation which is in evidence, apparent to the inspector, and capable of ascertainment by examination to prove that it is "actually existing." The Court, further, expressly states that when "physical items are estimated as worth so much new," depreciation is to be deducted, "if in fact they be depreciated," and presumably not otherwise. In the very nature of physical property, it is impossible after it has once been installed that full 100% can be represented by all the parts, except for the service intended.

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In the foregoing quotation from the Supreme Court's decision, the statement is made that depreciation must be deducted on the particular physical items being considered "if in fact they be depreciated." It will be noted that the Court expressly raises the question as to whether or not there has been depreciation, by the statement "if in fact they be depreciated." Absolutely, in every case, if market value —regardless of value for service purposes—were considered, depreciation of physical property would commence the instant it was installed, and there could be no "if" in the case. The use of "if" by the Supreme Court clearly indicates that depreciation should be deducted from reproduction new, only in case depreciation is obvious, or if the quality of service rendered has been impaired by use or by exposure.

If there is one thing that the decision of the Supreme Court in the Minnesota rate cases clearly establishes, it is that unwarranted assumptions, speculative bases, and hypothetical conclusions, will not be accepted as facts by this Court. This expression of opinion runs all the way through the decision, and clearly indicates that the determination of value by artificial rules, formulas, and speculation, instead of by examination of conditions, ascertainment of facts, and positive testimony thereon, will not have weight or carry conviction with the Court.

As the Supreme Court is unwilling to accept artificial rules, formulas, and speculation in determining value, as proposed by the Committee's Equal-Annual-Payment Method, and has repeatedly reaffirmed its decision that the present value of real estate is to be allowed, it will be seen that the recommendations of the Committee in these matters are inapplicable and misleading, and should not be endorsed by the Society.

The Committee apparently fails to appreciate the fact that the value of land is a capital account, not an operating credit or debit. If investors are to be allowed none of the chances for gain under advantageous circumstances, and must stand those losses accruing from disadvantageous conditions, the rate of return must be increased, but this is very difficult of accomplishment. If the investor were guaranteed, under all circumstances, a fair return, there might be some merit in the argument that he is not entitled to profits from the unearned increment, but such conditions do not now, and certainly will not, obtain in the United States in the near future; consequently, valuations proposed to be based on such premises are impracticable. If the utility investing in real estate is not to be allowed the unearned increment, the result will be ownership by individuals who will lease the required real estate to the utility at increasing rates, corresponding to the present value of the land, so that, in any case, the public will pay a fair return on the present value of the land.

The existing desiderata in utility valuation and accounting are directness and simplicity. Strict cost of reproduction applied to all

Mr. Floy. properties dispenses equal justice; has been approved by the highest Court; and is direct and fairly simple in its application when used by fair-minded engineers. It has come to be generally recognized as meaning the cost of reproducing new, a definite, existing, physical property on the basis of prices current at the time of estimate—and such estimate is made up to include everything that can be inventoried —regardless of original cost, age, service value, or present condition, as affected by depreciation. Reproduction new has not been accepted as meaning the estimate of cost at present-day prices, under conditions contrary to those existing at present and based on historic conditions. The Committee may consider that a combination of methods, namely, reproduction cost new and original investment combined, may be a fair method of getting at the existing fair value, but, for such method, the Committee should originate a new term and not attempt to legitimatize their innovation by adopting a generally accepted and widely understood phrase. If "cost of reproduction new," as defined by the Committee, is applied to long-established utilities, the resulting appraisal would not only be contrary to the decisions of the Courts, but would result in confiscation of private property.

When once determined, the cost of reproduction furnishes the basis for a uniform fair return to the investor, as well as the amount to be used in estimating the regular yearly allowance to be included in the rate charged for service to be paid by the consumer in order to protect the investment and maintain the service. The method of handling this latter payment to cover depreciation is a matter of detail, which can well be left to the public utility commissions, or other regulating bodies, but is not involved in the question of determining the present value of properties not heretofore under Commission rule.

It must be recognized that the investor in utilities has not put his money in to have it paid back in driblets, as proposed by the Equal-Annual-Payment plan of the Committee. The investor has placed his money with the expectation of a uniform return therefrom, on the part of the utility in which he has become interested; it is unfair and irrational to ask that he now receive back this investment in small, annual payments, which must be placed elsewhere, in order that the investor may receive the fair return on the amount which he considered his original investment in the utility. The gratuitous assumption of the Committee that, under its plan, the investment will be paid back yearly only in such amounts as may be required for additions to the property, is so highly theoretical, and contrary to what will be found practical, that such scheme cannot be seriously considered in the attempt to determine a generally acceptable basis of valuation and return between the utility and the public.

Mr. Whitten's book, which was of such great aid to the work of the Committee, is a masterful collection of Court and commission de-

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cisions. Mr. Whitten, however, like most writers, has been unable to avoid all errors, as, for example, his conclusions with regard to the Court decisions in the Cedar Rapids Gas Light Company case and the Des Moines Gas Company case. His personal views and opinions are sometimes "against the clear weight of authority," as stated in a recent brief of a well-known attorney. Because of the evident dependence placed by the Committee on Mr. Whitten's book, the following quotations from a review of that book by the Secretary of the Committee is of interest.*

"* * * The author is well qualified for the work undertaken by him, which has been done fairly and without apparent bias, though in his expressed personal opinions the author leans to the public rather than the corporate point of view in his constant tendency to revert to original cost rather than present value as the most important yard stick for valuation.

* * * * * * *

"With reference to the cost of reproduction the author concludes:

" 'Considered from all points of view the method of reproducing the existing plant under the actual physical and other conditions under which it was actually constructed seems fair to both parties. It is a rule that corresponds to the actual equities of the parties, while the other rule gives an unfair advantage, in some cases to the public and in other cases to the Company.'

"Here speaks the exponent of the public who seeks to enforce the doctrine of valuation by *original cost* rather than by the legal rule of *present value,* advocated by the corporation, and in force in all private and general business transactions. In the light of the past trend of court rulings, would not a fairer basis of determining reproduction cost involve the reproducing from the old original conditions of the plant in its present condition? The author's definition involves the reproducing of the work actually done by the company, not perhaps of the existing structure.

"The author's discussion of 'actual cost,' the possibility of its accurate determination, and its utility as a standard of valuation is certainly not justified by conditions in many of the older communities of the East.

* * * * * * *

"In the discussion of 'Pavement over Mains' crops up again the strong bias of the public-service commissions in favor of original cost, rather than present value, as a standard of valuation. If the reader will constantly ask himself the question, 'Is this original cost or present value?' in reading the author's own comments and quotations from no less an authority than Commissioner Maltbie of the New York Public Service Commission, in the discussion upon the paving question contained in paragraphs 168 and 169, he cannot fail to be struck by the inconsistency of the position taken. It is unfortunate, too, that the issue should be clouded by any reference to ownership of the pavement.

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* Review by Leonard Metcalf, M. Am. Soc. C. E., "Valuation of Public Service Corporations," by Robert H. Whitten, *Engineering News*, September 12th, 1912.

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"In the discussion upon 'Going Concern as Created Income,' the same strictures apply. The author prefers the Development Cost method of determination—clearly a reversion to original cost rather than present value. The author could not have been guilty of such a statement upon the reproduction cost method of determining going concern value as that contained in § 586:

"'* * * A public water supply is a necessity. A light system is a necessity. They are utilities that cannot be permitted to stop. The consumer must use them or suffer irreparable injury. *It is fanciful to speak of a development period for the new plant under these conditions.* If the old plant were wiped out, the new plant would have the business at once if already constructed, and otherwise, as soon as completed and ready for operation,'

"had he studied the actual experience of New Orleans, which has recently built new works, taken over an old small local plant, and in spite of drastic legislation requiring the abandonment of cisterns, is, in fact, *not in fancy,* slowly developing going concern value at substantial cost. So, too, his dicta upon going concern value:

* * * * * * *

"While the critic has no brief for either method and recognizes the inherent difficulties of the problem, the reproduction method of determining going concern value is the outcome of an honest effort on the part of the engineer to meet the requirement of the courts that the true reproduction cost shall be determined, and with all its possible inaccuracies, it is the only method yet devised which in any sense approaches such a determination."

The Committee has referred to the speaker's book, "Valuation of Public Utility Properties", using a quotation therefrom, as showing "the fallacy of the views expressed by engineers of much experience in valuation, that rates based upon the depreciated value of a property are necessarily lower than those based upon its full value." (Page 47.) As additional quotations from other writers are also given by the Committee, but without mentioning such writers by name, the exceptional honor thrust upon the writer would seem to be a little invidious. However, the quotations are from writers of such standing that to be classed with them and the Supreme Court in the condemnations of the Committee, offers no serious cause for complaint. The illustrations used by the Committee may disclose a fallacy in the methods proposed in its report, but they do not disclose any fallacy with regard to the practical matter of what is actually being done to-day in the valuation of utility properties and the fixing of rates based on such valuations. There are too many instances, known to those conversant with the subject, to need any citation here, in order to prove that the present theoretically depreciated value, where used for the purpose of fixing rates, results in a much lower return to the investor than if the full investment or the reproduction value new of the property had been

used for such basis. Therefore, the speaker's illustration does not seem to disclose anything in the way of a practical fallacy. Mr. Floy.

The soundness of the speaker's views as to the use of present value of land, inclusion of pavements over mains, and definition of cost of reproduction new, seemed to meet the approval of the Secretary of the Committee, at least, within the past year and a half, as shown by the following:

"The work of the commissions in determining land values is discussed in detail, as is the much debated question of the inclusion in or exclusion from reproduction cost estimates of the cost of paving over trenches, laid by the city subsequent to the construction of the pipe system by the corporation. The author points out the illogical position taken in this matter by some of our courts and commissions, and while admitting that in certain cases reproduction cost may not be a proper criterion of value, he soundly holds that in making an estimate of reproduction cost, the actual conditions of reproduction must prevail and the cost of the paving work be included, therefore, although not originally incurred by the corporation, as 'there exists no judicial pronunciamento for taking a part of the estimated cost of reproduction and leaving out the other part.' "*

The report (page 32) states that it is necessary to "adopt some practicable and equitable method of determining depreciation allowances" to meet certain requirements, which are given by the Committee. These stipulated requirements can be met much more simply and satisfactorily than by the adoption of the Equal-Annual-Payment Method. There is nothing in the requirements given by the Committee, which would prevent their being met by the use of either the Straight-Line, or the Sinking-Fund Method; which of these methods is preferable, on account of other considerations, need not be here considered.

One definite purpose of the Committee is apparently to provide equal annual payments by the rate-payers, continuously; and yet, as shown by the table on page 34, these payments are not equal, unless the same rate of depreciation and return is used. In order to attain this feature, which, in the minds of the Committee, seems so desirable, one of its members in a recent paper† states:

"I should like to suggest that the interest be the same on capital and depreciation allowances and that profit or wage be computed on cost of operation."

In order to have the report of the Committee acceptable to utilities, any requirement, that the rate of depreciation and rate of return should be the same, must be waived, and, as the greater the difference

* Review by Leonard Metcalf, M. Am. Soc. C. E., "Valuation of Public Utility Properties," by Henry Floy, *Engineering News*, September 12th, 1912.

† "Suggestions for Public-Utility Rate Making," by W. G. Raymond, M. Am. Soc. C. E., *Engineering News*, March 5th, 1914.

Mr. Floy. between these rates, the greater the inequality between the early and later years, during which the rate-payer is making payments for the service rendered, the recommendations of the Committee with regard to the treatment of depreciation must fall of their own weight.

Without discussing minor inconsistencies and other details with which the speaker cannot agree, it would seem as if the Committee was attempting to determine only one basis of "cost," which it considers should be used as the "value" of a given property. The Supreme Court requires that several bases of cost should be considered, and, by an act of judgment, the fair value then determined. It is on this account that the Committee's recommendation, mixing original-cost and original-investment methods, with reproduction-cost-new methods bring about a result which does not conform with any one of the valuations recognized by the Supreme Court. Such confusion is the cause of the Committee's conclusions that unless expenditures (many of which are not now ascertainable) have actually been made in connection with original construction, allowances therefor should not be included in the present value (page 28).

The recommendation of the Committee (page 43), classifying depreciation of property having a life of less than 5 years, as maintenance, and that having a life of more than 5 years, as replacement, is not progressive or in accordance with the best practice. The practical difficulty of separating maintenance from renewal accounts is generally recognized, and some utilities, with the approval of public regulating bodies, are now setting aside annually a lump sum to cover maintenance and renewals. Against this gross sum, the current year's maintenance and renewal expenditures are first charged and the balance of the appropriation is carried forward as a depreciation fund, for future renewals, thus simplifying the accounting, avoiding controversies between the representatives of the utilities and those of the regulating bodies, and insuring that the public receives full credit for all payments on account of depreciation.

The Committee proposes to depend mainly on calculated theoretical methods in determining depreciation, which do not give due regard to the condition of maintenance and repair in which the property may actually be found—regardless of its age. The Committee argues that the utility is only entitled to such allowance or depreciation as may have actually been expended (page 42), apparently failing to distinguish that the rate allowed for depreciation is entirely independent and distinct from the state of depreciation in which a property may be found to exist. Maintenance expenditures may almost wholly, if not entirely, offset losses from deterioration. Depreciation is but the sum of maintenance and renewal, and sufficient may be spent for these accounts to equal the full amount of deterioration.

Mr. Hendricks.

V. K. HENDRICKS,* M. AM. SOC. C. E. (by letter).—It seems desirable that the Committee should discuss more fully in the report the treatment of the enhancement in land values. As the writer understands Paragraph 19, Appreciation, on page 73 of the report, the public service corporation is to repay to the rate-payer the amount of the enhancement in values of land. As brought out in previous discussions, all other owners of property enjoy the benefit of such enhancement without having to pay for it, and the writer does not see why public service utilities should not be treated in the same manner.

Even if it were proper that the corporation should pay for the enhancement in value of land, there seems to be no way by which this payment could be made to the rate-payer at the time of such enhancement. Changes in land values are erratic, and there is no uniform change on which it would be possible to base any reliable calculations. If the public were paid for this enhancement by direct donation to the State or Federal Government, the payment would not be made to the rate-payer, but would be made to the general public. Such an adjustment would necessarily have to be made periodically, after the enhancement is known, and would be in the nature of a construction charge (or credit, as the case may be) and not in the nature of a refund to the rate-payer, or reduction in the rate.

As the writer sees the matter, the public service corporation should be entitled to the full benefit of enhancement in land values, and, even if this view is wrong and the Committee's view is correct, it seems impracticable to treat such appreciation "in the same way that the depreciation of the perishable property is treated, but on the opposite side of the account."

Mr. Wilgus.

WILLIAM J. WILGUS,† M. AM. SOC. C. E. (by letter).—Since the preparation of the previous remarks by the writer, the Committee has issued a supplemental report, which is summarized in the following paragraphs:

"It has already been shown in the foregoing discussion that, in connection with the Replacement Method and the Sinking-Fund Method of providing for depreciation, value new must be used in order to do justice to the owner of the property, and, for the reasons given, it would obviously be confiscatory if the value in such cases were to be depreciated from year to year.

"In connection with the Straight-Line Method of providing for depreciation, it has been shown that equity requires that the depreciated value be used. When this method is used, the rates necessarily decrease from year to year, the amount of decrease being small in the case of short-lived property and large in the case of long-lived property, but this decrease does not represent injustice to the owner of the

* Previous discussion by Mr. Hendricks appeared in *Proceedings* for February, 1914, on p. 850.

† Previous discussion by Mr. Wilgus appeared in *Proceedings* for February, 1914, on p. 366.

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property, as he will receive rates which, taken as a whole, during the life of the property, will be equitable. By the use of this method, he would receive too high rates when the property is new and too low rates when it is old, with a correct average amount.

"The Equal-Annual-Payment Method has been devised to obviate this difficulty, and when it is used in connection with the depreciated value of the property, as it must be to produce equitable results, the rates will remain constant regardless of the age of the property or the extent to which it has depreciated."

On railroads, except in the case of equipment, the restoration of depreciation of each item of property as it goes out of use is charged to current expenses, or, in some instances, to profit and loss surplus which may be looked on as the equivalent of a sinking fund; and therefore the methods referred to in the first paragraph of the quotation would apply thereto. Equipment renewals in part are charged to reserves, for which no standard practice prevails, some roads adopting very low rates and others comparatively high ones, all depending on the character of maintenance and the viewpoint of each company.

Utter confusion would follow any attempt to use both of the first-mentioned methods, and the third is believed to be inapplicable to railroad practice. Hence, the first method, of which cost-of-reproduction-new is the basis, is believed by the writer to be the only practicable and logical one to adopt, ordinary current depreciation chargeable to expenses to be defrayed out of current earnings, and deferred depreciation likewise chargeable to expenses to be provided through an allowance in the rate that will fairly compensate the owner for his assumption of the liability of making good such depreciation, as the need arises, without swelling the capital on which the public is expected to yield a fair return.

It is to be hoped that the report of the Committee, in its final form, will bring out clearly and forcibly its later views, which recognize that confiscation would follow the deduction of depreciation from cost-of-reproduction-new where the rates, coupled with the accounting regulations of the Interstate Commerce Commission and other regulatory bodies, do not provide for a gradual restoration of capital to the owner.

Mr. Dana.

RICHARD T. DANA, M. AM. SOC. C. E. (by letter).— The great weight which this report will have in the development of valuation practice, and in legislation connected with it, makes it of importance to eliminate, as far as possible, contradictions in terms and ambiguous interpretations of the fundamental facts that must govern any theory of this subject. Valuation is the subject of the report, and there are at least several kinds of value, yet the Committee seems not to have considered it necessary to define the term, value, which it has used in several senses. Most laymen fail to appreciate the fact that confusion

Mr. Dana.

in the use or meaning of this word is far more likely to result in injustice, or inequity, to the owner of a public utility plant than to the public, as will appear in the following.

Just as it has always been standard practice among engineers "to give the contractor the benefit of the (reasonable) doubt", so should it be the duty of appraisers to see that the owners of property receive rates which shall afford them a reasonable return on their property, and as long as the community presumes to substitute its own authority for the law of supply and demand, where free competition is not possible, just so long should it lean, if any way, toward the owner, in the determination to guard him against loss of his property by inadvertence and "without due process of law". It may be objected that the rights of the public must be conserved, which is true; but it is a common-law principle, centuries old, that before depriving a man of his property or of his liberty, the burden of proof is always on the community or the plaintiff. In a criminal action the State must prove its case beyond a reasonable doubt, but in a civil suit the plaintiff need only prove it by the weight of evidence. In these matters of rate-making the same broad principle of common-law justice, which for hundreds of years has maintained the sanctity of individual rights in all community matters, must sooner or later crystallize the practice into the same general form. The burden of proof should be on the community in all matters that are open to serious doubt as between it and the individual or corporation. The trend of practice is now plainly in this direction, yet many errors have been committed through failure to observe this rule.

The universal method in valuations is by synthetic estimates, in as great detail as possible. Now, the great fault of this method—perhaps its only fault—is that the likelihood of omitting items which should be included is far greater—a hundred times greater—than the likelihood of putting in items which should not be included. Consequently, every appraisal, other things being equal, is more likely to be too low than too high. The inclusion of a considerable percentage for omissions and contingencies is an effort to compensate for this inequity. Consequently, in a standard appraisal it is reasonably certain that the net amount of the omissions, as reduced by the allowance for omissions and contingencies, will be in small proportion to the whole amount of the valuation; but because there is to-day great confusion as to the meaning of the term, value, an owner may be deprived of a very large part of his property without compensation. The Courts in general will allow a "fair return upon a fair value" (page 6),* and it has been held (page 20),* "that the value of the property is to be determined as of the time when the inquiry is made regarding the rates". The Committee in its report infers that this implies that present (as of the time of the appraisal), rather than original, unit prices, should

* Committee's Report.

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be used in determining the cost of reproduction. Does this necessarily follow? The whole matter hinges on the meaning of "fair value", and the writer purposes to show that, with a logical definition of this term, the Committee's inference is untenable. The value of a plant is its intrinsic worth to its owner. Measured in money, it is "that sum which he could exchange it for without intrinsic gain or loss". As ownership exists for the purpose of periodical return (interest or dividends), the theoretically perfect method of computing intrinsic worth would be by capitalizing net earnings, but this is impossible for rate-making purposes, involving reasoning in a circle, as has been often pointed out.

The application of the definition of value just given affords a convenient criterion for testing the method of computing unit prices. By way of illustration, assume a private water plant, 60 years old, with a large reservoir excavated originally in earth. The earthwork cost probably 40 cents per cu. yd. by hand methods (old rates of wages), but could be done to-day for 30 cents (steam shovel and present rates). If the owner should exchange this reservoir for money, on the basis of 30 cents per cu. yd. for the excavation, he must stand a loss of 10 cents per cu. yd. It cannot be objected that the cubic yard which originally cost 40 cents to remove has depreciated "functionally" or "naturally", unless another reservoir could be dug for less cost or for the same cost with greater usefulness. It cannot be objected that he lost the 10 cents per cu. yd. as soon as the steam shovel was developed, and that the community is not responsible for what he lost before the appraisal, because he has not lost it until he parts with it, as long as it is useful to him in his business. If 10 years or 2 years before the appraisal the reservoir became useless, then at that time, he made a loss of the full cost, perhaps. If it deteriorated functionally, then there was a partial loss by depreciation; but with such functional depreciation, which depends on capitalized net earnings of the depreciated plant in comparison with those of a standard plant, the steam shovel has nothing to do. The cubic yard which was taken out by hand 60 years ago is bringing him the same income as if it had been excavated with a steam shovel. As it could not have been excavated with a steam shovel in those days, its actual cost represents the fair equivalent of what the owner has invested in it. If he receives less than this for it in exchange, he stands a loss. If he receives more, he gets a profit. If the community allows him 30 cents for it, it compels him to lose 10 cents, and deprives him of his property without compensation. He might be willing to sell it for 30 cents and take a 10-cent loss in order to go into a more profitable business, but the community has no right to make him sell it and take such loss.

From another viewpoint, consider a telephone company which owns, among other things, 100 000 000 lb. of copper wire on poles, for which it paid a weighted average price of 15 cents. Its investment in the wire will then be $15 000 000. Now, suppose that at the time of ap-

praisal the price of copper wire was 29 cents. If allowed rates on the basis of 29 cents, the company would get a profit of $14 000 000 on that item. It cannot be argued that the company made the profit as soon as the market changed and the open market price rose to 29 cents, as it reaped no benefit from the market rise if it could not sell its wire, but kept on telephoning over it. A profit sans benefit is of infinitesimal value. The company would have made a profit if it had sold at 29 cents, but had it done this, it would not have had the wire over which to telephone. The community, in taking over this wire at any other price than that which was paid for it, must either compel the company to take a loss or allow it to make a profit, neither of which alternatives is consistent with a fair "value". There will be some copper wire in the warehouses, say, 10 000 000 lb., which is not in use, is not earning anything, cost 15 cents, and could be sold in the market for 27 cents net. It does not have to be kept on hand for emergency repairs, but is held pending some proposed new construction. It differs from the wire on the poles in that it is exchangeable for cash at 27 cents without interfering with the operation of the company as a going concern. Its owner can sell it the day before the appraisal and "take" a 12-cent profit. This he cannot do with the wires on poles. If the community allows the company 15 cents for the wire in storage, it will compel it to part with the profit which it earned by speculating in copper (which is what it did when it bought in advance of its wants), and, therefore, it would be causing the company a loss on the exchange, which is inconsistent with the definition of a fair value. To avoid intrinsic gain or loss, the community should allow original unit prices for items which are not marketable by the owner without going out of business; and market sale prices for such items of plant as can be disposed of without interfering with the business. A corollary to this rule necessarily must obtain when a loss has taken place or a profit has been earned on plant in service. Mr. Dana.

It is possible for the owner to suffer a loss or gain a profit on the plant without having a free market for it. Depreciation entails a loss to the owner, and when it is a function of the lapse of time (natural depreciation) to avoid this loss, it must be compensated for, either by a sinking fund, to increase *pari-passu* with the depreciation, or by an equivalent amount invested in some other manner, such as other plant (renewals), etc. On appraisal, the depreciated items always entail a loss which was incurred by the owner before the appraisal and with which the community has no concern, as such loss is not a result of the appraisal. If the owner has kept his investment intact by a depreciation fund or by purchasing other plant or investing in some other manner the equivalent of the depreciation sinking fund, then the value of the total assets remain as they were, but the part of the plant which has depreciated has already stood the owner a loss. The community must allow its depreciated value, in rate-making, else it would be

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standing part or all of the loss already incurred by the owner during the years of operation, which is against the hypothesis in the previously given definition of value. In relation to this matter, two points must be very carefully noted: First, that natural depreciation is a loss of value, proceeding gradually, which cannot be determined by market conditions, nor even by inspection of the plant, but must be calculated theoretically on certain assumptions of life, age, and interest rate; and second, that the capital may be preserved intact by a separate depreciation fund to be set aside (a very rare occurrence in practice), or by investing an equivalent amount in other plant or in some other asset, or again by a part of the potential increase of business caused by the money spent in development (or not paid in interest or dividends during the early years of the business) and which has been termed "development expense". Considered in this manner, there is no difficulty about constant rates and constant dividends, without impairment of capital and without departing from a consistent definition of value. If the owner has foolishly wasted money in accordance with bad practice, he is not entitled to have it covered by development expense, as it is not a proper expense of development.

There remains to consider functional depreciation. Consider a 30-year, single-cylinder, non-condensing, steam engine belted to a modern generator. Assume the engine to be in perfect condition and functioning as well as on the day on which it was first placed in service. It is transforming steam power into mechanical work at a certain rate and at a certain unit cost for that function. Nothing has been spent on it in that time except for care, housing, and ordinary repairs. It could not be sold except for scrap. No fund has been set aside with which to buy a new engine in its place. A modern engine of the same capacity can be bought, which will transform steam power into mechanical work at a unit cost which is to that for the old engine, say, as n is to m. These unit costs do not include interest, etc., on the prices of the engines. The new engine could be bought and erected for N dollars. The amount which represents the fair value of the old engine is that figure which will make the unit cost of transforming energy by the old engine the same as that of transforming energy by the new engine, if bought. Otherwise, if the new engine can transform at less unit cost, the owner could exchange and make a profit; but, on the other hand, if M be such a figure that the interest charges and operating expenses of the new engine would be greater than for the old, a compulsory exchange would result in a loss to the owner.

Let the cost of the new engine, connected. . . . . $= N$
" " value of the old " . . . . . $= M$
" " unit cost of transforming power by the new engine $= n$
" " " " " " " " " " old " $= m$
" " number of units of power transformed per year. . $= U$
" " interest rate on money. . . . . $= r$

Then the annual cost of operating the new engine would be $Nr + nU$..(1) Mr. Dana.

" " " " " " " old " " " $Mr + mU$..(2)

" " unit " " " " new " " " $\frac{Nr}{U} + n$...(3)

" " " " " " " old " " " $\frac{Mr}{U} + m$..(4)

By the conditions of the problem these must be equal, and, therefore,

$$\frac{Mr}{U} + m = \frac{Nr}{U} + n, \text{ whence, } M = N - U\frac{m-n}{r} \ldots\ldots (5)$$

It can be shown that this is the same value for M which would follow from equating the capitalized annual cost of operating the two engines.

The owner of a certain water plant thinks that it is worth more than $100 000 to him; the mayor of the city thinks that it is worth $50 000 to the city which wants it and has the right to acquire it on a valuation by disinterested experts. The city would scrap a large part of it and have an entirely different kind of works, and considers itself justified in allowing scrap value for all the parts of the plant that it would like to scrap, on the ground that the new plant would pay for itself in a few years out of increased net earnings. The city's adviser estimates that for a certain pump the value of M is actually negative. On this theory it would be consistent to allow the owner less than nothing for the "value" of his pump and of any other plant which, by this formula, will be less than nothing. The solution may be found by applying the criterion that "value is that sum which can be exchanged for the plant without intrinsic gain or loss to the owner". The pump is raising water which is being sold for more than the operating costs. The owner is receiving a return on the money which he has invested in the pump and the remainder of the plant, and he is now receiving some return on money spent in developing the business, by way of Development Expense. The pump in question is earning money, and it, or some other pump, is essential to enable the owner to operate the business. Manifestly, to take the pump away from him without paying for it would entail a loss to him. He admits that he could earn more by buying a new pump, but he has put so much capital into the development of the business, that he does not wish to invest more money at present. The money spent in development has been wisely expended, because the net earnings are now nearly enough to pay a reasonable return on all that he has invested in the water-works, and the net earnings are growing so rapidly as to promise in a few years a profit on the investment, when he will be in position to secure plenty of additional capital for improvements, etc. Now, the city does not contemplate buying from him just the pump

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alone, at a negative valuation. It has an option on the whole plant of which the pump is but one part, and he stands no loss if he receives a negative value on the pump, provided he gets a proper positive value in the form of Development Expense. If it should appear that he has received a fair return in dividends and interest on the capital invested to date, operating expenses to include depreciation, then there is no Development Expense, and the pump no longer represents to him a capital investment equivalent to its original cost.

In calculating the Development Expense proper depreciation charges should be considered a part of operating expenses, in arriving at the correct amount of net earnings. On that basis, and that basis alone, can the Development Expense be correctly figured, and then it is logical, consistent, and proper to allow to the items of plant their true functional value, which may be, and often is, negative. Where this practice is omitted, injustice is sure to be done. When properly applied, it affords the owner ample protection against actual loss, and, at the same time, enables the community to acquire the plant or fix the rates on a basis no higher than would have been the case had the community built and developed the plant in the same manner and at the same time that the owner actually did develop it. Moreover, as the development by the owner is likely to have been more economical and efficient than if it had been done by the community, there is a greater or less intangible profit accruing to the latter if the owner is allowed only the exact amount represented by his investment. To regard the owner as having been in effect the agent of the community in developing the property, besides being fair to the owner, is not unfair to the community. On this criterion is based the so-called "Agency Theory" of valuation.

Very frequently, and nearly always in the case of railways, it occurs that competitive rates are such and development expense has been so heavy that the owner is not receiving a normal rate of return on his capital, nor can he perceive such a return in sight. He, therefore, would be glad to sell his property for less than the "fair value", taking a loss in order to go into a more profitable business. If the community can ascertain what this amount is by bargain, then a purchase at that price may be effected, but obviously it would be the height of injustice to regulate rates so as to cut down the net return below a normal rate on the owner's capital in order to coerce him into selling at a loss. By the same token, his point of view of value is that which should be taken in fixing rates.

Aside from these reasons for using original rather than present unit prices in computing plant value, there are occasional difficulties in the practical application of the present-unit-price method. For instance, in Central New York State a good many piles, driven 50 or 60 years ago, are of the most magnificent white oak, with butts of

22 in. and more. There is no market for that kind of material at present in that part of the country, and freight from a long distance would make the unit prices on such material prohibitive to-day. To apply present unit prices then on such material is virtually impossible, except by assuming that the owner should be contented with the unit price on something else "just as good". Consider the matter of ashlar masonry laid in Portland cement mortar when the cement cost $4 per bbl. To-day, the same function would be performed by concrete made with cement costing less than half that price. To allow the owner present prices on concrete and calculate his masonry in that manner, in fixing rates, would compel him to stand a very considerable loss. It cannot be objected that he must stand that depreciation anyway, because the original cost of the ashlar with the high priced cement must be depreciated to get the present (to him) value, as outlined. To allow present prices of cement and calculate the present reproduction cost of ashlar, would force him to take an unfair loss on the cement, and so little ashlar masonry is built in these days that there is hardly a free market price for it. It is in a class with the large oak piles previously mentioned. Mr. Dana.

The writer understands from the report (page 15) that it is the opinion of the Committee that though present prices should be used, the estimated cost should be based on the conditions existing at the time of construction. As prevailing prices are among the conditions of construction, the Committee's ruling seems to the writer to lack consistency, especially as

"The Committee believes it to be a general basic principle that where money has been expended with due discretion * * *, the company is, in the case of a newly erected normal property, entitled to a fair return upon a value which, subject to modifications, equals the amount of money invested."

With the last paragraph under "Unit Prices" (page 20), the writer cannot agree. In the case of lumber, as quoted for illustration in the report, the prices have been rising, and probably will continue to do so in the East, until the Panama Canal is finished, when they will go lower in the East and higher in the Far West. Can the building of the Panama Canal afford an excuse for cutting the rates charged by a railroad in Pennsylvania, which has been able to earn just enough to pay interest on its actual capital investment, with depreciation on plant which includes many timber trestles? To depend on the appraiser "to select the length of the period with a view to obtaining a fair prevailing price" does not seem logical or practical in its application. The writer ventures to call attention to the fact that, in the vast majority of cases, the aggregate depreciation is much more than the aggregate appreciation on physical property, and that this fact is likely to militate against the fair interest of the owner, unless a correct

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amount for Development Expense be allowed. Now, this line of reasoning assumes that there is a positive "Development Expense", and is consistent with the "Agency Theory" of valuation, to which four very important objections (among others), have been made, *viz.*:

(1) That the owner never was an agent and, therefore, should not be considered as such;

(2) That the owner is entitled to appreciation in value, in such items, for instance, as real estate, arising from the development of the community in which his property is located; and that the theory outlined does not allow him such profits;

(3) That when the rate of allowable return is taken from the standpoint of the owner, the community is overcharged, because it could have obtained the money at much lower rates than the "fair-return" rate to the owner;

(4) That the Agency Theory eliminates Franchise Value.

(1) The first of these objections, which has been made in legal proceedings, is somewhat overstated. Had the owner actually been appointed agent for the community, the latter would have to stand for all his blunders, except where intent to defraud was demonstrable. Frequently, the community treats the owner as anything but its agent, as in the case of Seattle, Wash., holding down his rates by commissions and competing with him by a municipally owned plant. Whether or not the owner actually was an agent seems to be beside the issue, except on the basis of a purely legal technicality, which need not concern us here. The Agency Theory has been thus called because the term conveniently expresses a logical hypothesis which, in its proper application, protects the owner against actual loss, except by blunders, as though he were the community's agent.

(2) The second objection is of much importance. There are cases in which development expense has been extinguished by surplus earnings, although this is rare indeed in the railroad business.

Where the land can be sold and the profit gained without interfering with the business as an operating entity, it seems that the owner is entitled to the profit, as before discussed. In regard to the property which is an integral part of the business, the Courts have ruled that the owner is entitled to appreciation, he is taxed on the appreciated value, and if he be not allowed to earn a return on the appreciated value, what justification is there in making him take a loss on property which has been lost or has depreciated, such as an abandoned piece of track?

Were a possible competitior to appear and attempt to buy the owner out, there is little doubt that the competitor would expect to pay about the present value of the land. If he succeeded in getting it for less, he would consider it a fine bargain. If he had to pay much more he

would buy elsewhere, or give up the idea. Now, the community, under the common-law theory of individual right, has no right to compel the owner to take less for his land than a competitor would expect to pay. If there is other land available, the community can buy the other land and allow the owner to use his land in some other business or otherwise dispose of it. It might not want to do this, because of the expense of moving tracks and buildings to another site, but that is no reason for not giving the owner his profit, if he is going to be forced out of business. In such a case as that of the New York Central Terminal on Manhattan Island, there is no other available site for the purpose, but here the owner must be entitled to a value based on the assumption that he may proceed to enjoy equally desirable land if he should want to go into some other business or otherwise dispose of his land, turning over the old business to the community. For rate-making purposes, the rates must be adjusted on the assumption that the owner would fare as well as if the community bought the property on the fair valuation. Any other view than the one stated will make the owner wish that he had used his property in some other business, one in which the community would have left him in peace to enjoy his prosperity. Emphasis should be laid on the fact that the "greatest good to the greatest number" is consistent with placing a premium, rather than a discount, on the use of private capital for public purposes. Any other view means socialism, if not anarchy. Mr. Dana.

(3) In regard to the third objection, the writer believes that if the owner put in his own capital, he has a right to the same return on it as if he put it into another business. The community might have borrowed it for less, but it did not, and as it had its choice in the matter and the owner did not, he is entitled to the normal rate of return on private capital.

(4) The fourth objection has certain intrinsic features in common with the second, and furnishes a clue to its solution.

If the owner is to be paid rates which will just allow him to earn a fair return on his investment, and there is no prospect of his being able, or allowed, to earn more than this in the future, of what good is a franchise to him? The original and only idea of a franchise or charter has always been to induce him to apply his money to public purposes rather than to use it in some other form of industry. Now that the State has money of its own, it seems hardly equitable to penalize the owner because he allowed himself to be persuaded to invest it under the inducement of a franchise or charter. Where there has been expense of development, which has not been amortized out of net earnings, and is not likely to be, there is no "franchise value" anyway, and the "franchise" simply amounts to a contract on the part of the community to let him stay in business and try to make up his losses. Where there is no accrued development expense a franchise

Mr. Dana. is a public bonus, in the form of a contract, whereby the owner may expect to earn an extra return on his capital as an inducement not to go elsewhere with it.

These four objections to the Agency Theory are based on what has been termed the Competitive Theory, which sometimes gives a total appraisement higher than the Agency Theory, and sometimes lower. In general, where there is a considerable Development Expense, the Competitive Theory gives a lower total than the Agency Theory.

It is often necessary in an individual problem to make two or more valuations, one on the Agency Theory, and another on the Competitive Theory, in which case the totals will not agree. It is certainly illogical to make a computation of value, some of the elements of which are calculated on the one theory and the others on the other.

Mr. McCormick. R. S. McCormick, M. Am. Soc. C. E. (by letter).—The Committee has presented a very able and exhaustive report, and it will doubtless be received as expressing the Society's stand on this subject.

The writer, however, desires to call attention to the fact that, in his opinion, the Society cannot afford to go on record in this way, and although the Committee has stated that the report is incomplete, and doubtless discussion will bring out points not touched on, he feels that it would have been more consistent with the object and aim of the Society to have confined the report to the question of valuation alone, omitting the application of such valuation, at least, to the extent of applying the whole argument to this purpose. The report might have been treated as an abstract subject of valuation, with and without depreciation.

There is a difference between the valuation of railroad property and other kinds of public utility property for rate-making purposes. Competition is not dead in the industrial world, not even as between large combinations, and although this competition is of a different kind from the old sort, it is not the less real, and its effect on the fixing of rates is a live issue between traffic managers on railroad systems. Where this kind of competition is possible, the matter of rates will never be wholly, or probably in any great measure, based on the principle of a fair return on the value of the property. This principle, however, is equitable, and, where it is possible to apply it, will give equitable returns. In the case of public utilities such as water-works, street railways, gas and lighting plants, etc., it is possible so to base rates. In the writer's opinion, however, it is wholly impracticable to apply this principle in this way to 95% of the railroad mileage of the United States; and, to say that these properties are valued for rate-making purposes is a misnomer.

The valuation of railroad properties, no matter how made, or on what principles as regards the question of depreciation, can be useful

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only as a guide to prevent exorbitant rates, and can never prevent lower rates than will give a fair return on the investment.

In the case of local public utilities, on the other hand, this valuation can be the fixed base for rate-making, because, speaking generally, there is no competition. For this purpose only a discussion on valuation for rate-making is properly applicable.

To value properly any one of the great railroad systems involves a most intricate and exhaustive examination by men properly qualified, and to know when an equitable value has been given to the properties is well worthy of the best efforts of those engaged in the work.

When all the data are collected and reported to the Commission, and a value is placed on the multitudinous items comprising this inventory, its value for rate-making purposes can be a fit subject for discussion.

The writer is of the opinion that such a valuation should be for the sole purpose of ascertaining the worth of the properties as property, keeping in mind the use to which it is put, and should have no bearing as to the purpose for which the valuation is made. This value may be present value, original cost of property to date, or a value based on cost of reproduction with or without depreciation. The various differences of opinion as to which method will give the true value presents a sufficient field for discussion without including the purpose for which it is to be made. It is unwise to go at this subject with any other object in view. This is especially applicable to railroad property, and the Interstate Commerce Commission has begun this work under a most serious handicap because the railway companies are getting a wrong impression in regard to its object.

It will not be possible to carry on this work and do justice to the carriers unless the engineers of the Commission and the companies' engineers agree on a common basis regarding values. To hamper either side with the burden of reconciling these values for any particular purpose is essentially wrong in principle, and, at least, should not be a subject for discussion until the inventory of the properties is complete and the value fixed by the Commission under the terms of the Act. Section 19-*a* certainly is explicit as defining what the Commission is to do, and it does not say that this valuation is to be made for the purpose of rate-making.

Although it is understood that such at least is one of the objects of the valuation—perhaps the principal one—the Commission has no authority to make this valuation with this end in view, and hence the Committee's report should have dealt with the subject in a broader sense, or, if this purpose—rate-making—was the object, it should have recognized the distinction between public utilities of the class where absolutely no competition is possible, and those where it is possible. This competition, although perhaps not fixing the rate, has, in a

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broad sense, much more to do with it than the actual value of the property could ever have. The physical value is of great assistance in adjusting rates, but not in making them.

Mr. McCarty.

RICHARD J. MCCARTY, M. AM. SOC. C. E. (by letter).—Realizing the importance of light on valuation of public utilities, the Board of Direction in October, 1911, appointed a Committee to investigate and report on the subject. At that time the full significance of the problem had not been developed, and it was generally regarded as one to be solved in the main by the Engineering Profession.

To judge from its Progress Report, the Committee addressed itself to the task with the utmost energy, and did great credit both to itself and to the Society by the public spirit manifested.

The passage of the Federal Valuation Act, and its approval on March 1st, 1913, made the problem one of national importance, and caused its fundamental significance to be investigated by many who before that time had given it particular attention only as applied to special cases.

A thorough consideration of the general character of the problem shows that its ultimate solution must be reached from the standpoint of Economics, Law, and Equity, rather than of Engineering. That the Committee realized this is shown by the following quotation from its report:

"The principles and methods involved are now being carefully considered by many public service commissions and the Courts, as well as by engineers, lawyers, and committees of various societies having to do with public utilities. The subject is clearly in the developmental stage, and any conclusions reached at the present time must be considered as tentative."

The Committee then states that:

"It is recognized, however, that the whole subject is in a developmental stage, and while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

It is not here held that the problem of valuation does not call for the highest type of engineering ability, but it is held that the work of the engineer terminates at the point at which the quantities and values for use of the different parts of the physical property of any given public utility shall have been determined, and that from this point forward the problem ceases to have particular significance to him. The reason for this is that the determination of the value for use of physical parts of a public utility, and the application of such values in determining the amount of capital on which a public service corporation is entitled to a fair rate of return, are problems of an

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entirely different character, and belong to entirely different departments of human thought and experience.

This is recognized in the Federal Valuation Act itself, which requires that there shall be ascertained and reported the original cost to date, cost of reproduction new, cost of reproduction less depreciation, other values, elements of value, and, separately from improvements, the original cost and present value of lands, rights of way, and terminals. The Act, however, is so far from stating the manner in which this information shall be applied, that it does not even give the purpose for which it is to be used.

The manifest intention of the Act is to obtain certain information through the Engineering Profession concerning the physical features of public service utilities, and to use the results thus obtained as a basis for determining the amount of capital on which the owner is entitled to a fair rate of return, and for other proper purposes.

For the American Society of Civil Engineers to investigate and give to the Engineering Profession information and data concerning the determination of the value for use of the physical parts of public utilities is eminently fit and proper, but for it to go beyond that point would be an invasion by the Society of the respective provinces of Law, Equity, and Economics.

This, the writer is convinced, the Society cannot afford to do, either in justice to itself, as the leading engineering organization of the United States, or in justice to the public, which heretofore it has benefited to such a great extent.

The effect of an attempt by the Society to solve problems of Law, Equity, and Economics would be great, because of the great influence on the Governmental authorities, and on the Courts, of any conclusions which it might reach or apparently endorse, and inasmuch as the Committee itself had recognized that any conclusions reached at the present time must be considered as tentative, it naturally follows that such conclusions should not be advanced under the auspices of the American Society of Civil Engineers.

The writer, therefore, is of the opinion that the report of the Committee should be confined strictly to the determination of the value for use of the physical parts of public utilities, and should not make any recommendations, or advance any tentative conclusions, concerning the manner in which the results of such determination shall be applied to questions arising between the owner of the utility on the one hand and the public on the other.

It seems particularly important that the Society, as well as the Committee, should adopt this course, because, it might easily happen that, owing to the influence of the former, and the standing and reputation of the members of the Committee, certain tentative conclusions might be embodied in the decisions of the highest Courts, which after-

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ward might be shown to be untenable, and thus work great injustice either to the public or to the owner of the public utility.

Mr. Buel.

A. W. BUEL, M. AM. SOC. C. E. (by letter).—Has the Committee considered what items should be covered by a "fair return"? If it covers risk of investment, why not also risk of development expenses or deferred dividends? It is necessary to answer these questions, in order to avoid duplicating the same items in the valuation and in the fair return. Is there a method by which a "fair return" may be determined from statistics? Although the final answer to these questions will have to depend on decisions of the Courts, it is the duty of valuation engineers to collect and present all data relative to the discussion.

Losing ventures would probably occur as often, if not oftener, under public ownership, and, therefore, they should be given some consideration. They may be divided into two classes with such approximation as can be ascertained by the historical facts. First, those which at their inception appeared to offer fair hopes of becoming profitable within a reasonable time, such as would attract conservative capitalists, but which failed to develop as anticipated; and, second, those which, at the time they were promoted, would not have appealed to prudent men as reasonable business risks. If the second class be eliminated, and the first class of losing ventures be combined with more or less profitable properties of the same kind, similarly conditioned, the expenditures for capital account and the net earnings or operating incomes for a series of years, for the entire group, should furnish a basis for determining the percentage of risk involved.

All the capital invested in the group, less the capital losses divided by the capital invested, gives the percentage of security. If a fair return, where no risk of loss is involved, is assumed at 4%, to compensate for the risk of loss of capital, it must be increased to the percentage that 4 would be on the percentage of security. Thus, if an average of 20% of capital invested in railway construction were lost, the percentage of security would be 80, and 4 divided by 80 is 5%, the return required to compensate for the capital risk.

To find the "fair return", however, we must also add compensation for lost and deferred dividends, unless these have been added to the cost of the properties in the valuations as development expenses. It would seem preferable to take care of these items as elements of risk in fixing fair returns, rather than to add them to cost of property as development expense, because they may vary greatly in the same territory and on competing lines which, both on the ground of equity and of the practical difficulties involved, require equal treatment on the common basis of the average return on the total capital investment in the group plus lost or deferred dividends, excluding losing ventures of the second class. It seems quite as logical to treat them as risks

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or losses as to consider them as development expense added to valuation costs. This method also has the advantage of greater clearness in segregating the values of purely physical property, thus reducing complications in arriving at operating values or valuations as going concerns, or, as some call it, good-will.

If we add all lost dividends, compounded at 4%, up to the average age of operating maturity, at which profitable properties begin to earn fair returns, to the capital invested in the group, then 4% on this sum divided by the percentage of security gives the fair return for this group.

Assuming 10 years for the age of maturity, for example, a percentage of security at 80 as before, also that the 20% of losing ventures of the first class just earn operating expenses and taxes on the average, and that the average earnings of the 80% of paying properties during the first 10 years is 2% less than a fair return, we have:

Lost dividends on 20% of losing ventures, compounded at 4% for 10 years, amount to 20% of 48.02%, or.....	9.6%
Lost dividends on 80% of paying properties during 10 years of development at 2% compounded at 4% amount to 80% of 24.01%, or.....	19.2%
Total lost in dividends.....	28.8%
Original investment in group.....	100.0%
Total " " "	128.8%

Four per cent. on 128.8% is 5.152%, which divided by 80 gives 6.44% as the fair return that insures an average yield of 4% on the investment. If this group represents four competing railway lines, the average return of 6.44% might be made up of 2% on the investment in one line, 10% on another line, and intermediate rates on the other two.

Of course, it is understood that these figures are merely assumed for an example, but it ought not to be impracticable to derive fairly close percentages for the losing ventures and lost dividends from a careful analysis of the records of large groups of kindred properties. It would probably be necessary to fix some limit on the age of maturity.

If the percentage of security is 90 and the lost dividends compounded at 4% amount to only 14.4%, then the fair return would only be 5.08 per cent. Of this, 4% may be called interest on investment and 1.08% insurance of capital and interest.

Though the Committee recognizes the equity of some recognition of operating values, or, to use its term, "efficiency", as a "modification of the physical valuation or of the rate of return upon such valuation", it in effect puts this problem up to the commissions

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and the Courts. Without question, some points involved in this element of valuation will have to await Court decisions for final determination; but, on the other hand, the larger part of this problem consists of highly technical matters which will devolve on engineers to solve. In fact, the determination of operating values or efficiency seems to be more especially an engineering problem than one of depreciation, which is largely a matter of accounting, or of inventory valuation of the real estate and many other elements of the physical property. The Committee's report suggests, quite correctly, the great difficulty of formulating any general rules for the practical application of this feature, but, because of its importance and the fact that it is generally so little understood, it deserves particular attention.

This problem of efficiency or operating values is so complex that it is extremely doubtful if it will be possible to lay down any formulas or rules except those of the most general character. Possibly, the only practical method will be found to consist in making comparative estimates of operating values of the existing property and of an ideal or hypothetical equivalent on some revised location or design, which involves surveys for revisions or new designs. But intelligent reconnaissance work will confine the extent of such investigations to practical limits. This method is not only feasible for determining operating values of railways and other properties with sufficient accuracy for all practical purposes, but it can also be used to advantage in estimating the efficiency of operating methods. Estimates of operating values and operating costs should be made in full detail, treating each variable item of cost separately for each varying condition. This is laborious and voluminous, but, for many cases, it is the only convincing method which as yet has been suggested. The writer has used it in studies of several thousand miles of railway locations and revisions under widely varying conditions, and it has always proved satisfactory. The degree of accuracy that may be attained by this method depends principally on the care used in finding correct data, particularly of the unit costs, and the formulas by which they vary. The latter, of course, can only be arbitrary approximations in most cases, but, if the data are based on broad averages taken from statistics of actual operations of properties working under as nearly similar conditions as possible, the final results will be substantially correct. No good would result from attempting a greater degree of accuracy than the variation of one normal year or another from the average, and the method suggested can be made to approach this if care is taken and good judgment used.

As the efficiency of any plant, location, or design depends to the greatest extent on its proper adaptation to the volume of business handled, it would be decidedly unfair to leave out of consideration the volume of business in sight, or that could have been reasonably

foreseen, at the time the property was constructed. The volume of business may have increased to such an extent that the original property or parts of it are practically obsolete, requiring reconstruction, revisions, and new facilities. No doubt many questions will arise as to just what is the equitable way to handle such cases of insufficiency of plant, but the circumstances and conditions may be so variable that a general rule could not be formulated to cover them. They probably belong in the class calling for the use of the best professional judgment available, except where they may be covered by Court decisions. Mr. Buel.

In another class are a few cases in which expensive revisions or reconstruction have been undertaken, entirely unwarranted by the volume of business in sight. For example, a railway with comparatively light traffic may have spent many millions of dollars in revisions to reduce mountain grades, without cutting down the summit elevation and at the expense of considerably increasing the distance, the savings in operating costs, being entirely inadequate to pay the interest on the additional investment. While, in time, the traffic may grow to a volume warranting the revision, if the expectation of such a result is deferred to an unreasonable time the revision should have been deferred. In such a case it is questionable what, if anything, should be allowed in the valuation for the cost of the revision. Though such cases are probably rare, some will be met. The example suggested serves to bring out by contrast the more numerous classes of poor original location or design and of those which have become obsolete and inadequate due to the growth of traffic, all of which suggest the large number of important and difficult questions in valuation still remaining to be solved.

J. E. WILLOUGHBY, M. AM. SOC. C. E. (by letter).—The subject under investigation of the Committee was worded so as to give prominence to "railroad property". It seems unfortunate that the report draws its illustrations principally from, and is pervaded with ideas relating to, water-works and sewerage plants. It is possible that the treatment would have been somewhat different had the underlying idea of the Committee been the valuation of railroad property. The writer's experience is confined to the construction, maintenance, and valuation of railroad property, and the comments hereinafter made are formed from experience with railroad property only. Mr. Willoughby.

The writer is not in sympathy with the formulation of principles and methods of valuation for rate-making, as distinguished from any other purposes. Whenever such a distinction is made, the presumption is raised that a valuation for rate-making purposes differs from, and is higher than, a valuation made as the basis of a sale. Such a presumption is unjust to the owner. The items which enter into the valuation covered by the Committee's report appertain almost exclusively to the determination of the cost (either original or replace-

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ment) of the physical property, and a valuation confined solely to the cost of the physical property is unjust to the owner. For example, take good-will. In a water-works monopoly, good will may be of but small value, but in a railway property it is often that which makes one railroad prosper, while the competing railway goes into bankruptcy. Inasmuch as the valuation of a public utility property includes other items than the cost of the physical property, the Committee's report is incomplete, and in that form it prevents the assignment of losses due to use (*e. g.*, depreciation) to the proper place in the valuation.

General Principles Involved in Valuation for Rate-Making.—As a corollary to the first of the two features suggested by the Committee as to the requirements of justice in any regulation of rates (page 7), the writer suggests that the annual return, in addition to covering interest and profit for the use of the capital, should provide such earnings as will enable the owner to meet the demands of the public through its public service commissions for additions to the property which cannot be capitalized without affecting the general credit of the owner. A familiar example of this kind of additions is found in the rivalry of the cities, and even towns, for union stations, monumental in character, and costing out of all proportion to the economic saving in transportation.

Fair Value for Rate-Making.—The fair value for rate-making for railway properties should consider the whole territory served by the properties. Rates that are not sufficiently high to provide an adequate return for some lines within a territory would for other lines in that territory, which enjoy a less capital expenditure and less operating cost, produce more than what is usually considered as a reasonable return. The Committee's report considers the abnormal case of a property which must be a losing venture, but overlooks the abnormal case of a property which must be a sort of El Dorado.

Methods of Determining Physical Value.—The writer is in accord with the Committee in its conclusions that a valuation of an old property must necessarily be made after the cost-of-reproduction method; and that the identical plant is to be reproduced and not a substitute plant. These conclusions are particularly true with regard to railway properties. If the cost of substitute railways were proposed, the actual locations on the surface of the earth of the existing railway under investigation, the rates of grade, and curvature, etc., with all the theories of economic operation, would be a subject of inquiry, the end of which would be confusion. Furthermore, reproduction of the identical railway prohibits the process of inquiry from theoretically correcting errors of location and construction, which often become apparent after the completion of the railway. This is just to the owner. Errors of location and construction are an inevitable consequence of railway construction, and the burdens of such rest on the public. The

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burden of an error of the owner in constructing a railway for which there is no economic necessity properly rests on the owner, except in those communities where the public through its laws and commissions, exercises the right of authorizing or denying the construction of any proposed railway.

Present, or Original, Conditions.—The conclusion of the Committee, that "the valuation should be based on the conditions existing at the time the various portions of the property were built, but on prices prevailing at or near the time of valuation", will result in confusion if applied to railway properties. Railway construction requires that there shall be the necessary materials of construction, machinery, teams, and tools of their several kinds, labor, skilled and unskilled, and transportation. The costs of construction work have a direct relation to all these items. When they are readily obtainable and cheap, the cost of the work is small; any scarcity of either causes the unit prices to advance. Some materials that were available (*e. g.*, cheap timber) when many of our railways were built, no longer exist; other materials (*e. g.*, cheap steel, explosives, and cements) were not then obtainable. As a result, many features of construction now called temporary were then adopted. It would be unjust to the public to capitalize in a present-day cost-of-reproduction valuation the trestles that were built long years ago and afterward filled, for a railway in a community where such methods are not at this day habitually practised as an economy. The convenience of transportation facilities to the railway subject to the valuation, affects enormously the unit costs of the work. The transportation conditions at the time of original construction, generally, were such that we cannot intelligently modify the unit costs attainable with modern excavating machinery to fit the original conditions, and we get unit prices and construction periods that are unreasonable. The difficulties recited by the Committee as making undesirable the use of present-day conditions are easier to overcome in railway valuations than the new difficulties that arise if we undertake to revive the original conditions. Take the Wachusett Reservoir. With present-day conditions, we would assume in case a highway or railway reaches one limit of the reservoir property, follows partly around the reservoir property and departs from it, that the highway or railway crossed the reservoir property, and was reconstructed on its present location, the cost-of-reproduction of which would be determined in accord with the principles and methods under discussion by the Society. An examination by borings would determine if the original soil were stripped from the reservoir site; and an examination of the depth of soil beyond the limits of the wetted area would determine the estimated depth of surface soil removed. The damages paid to the adjoining real estate, as incident to construction, would be estimated in accord with the experience of the inquiry as against all

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real estate in the condition of improvement which that real estate exhibited at the date of the inquiry. If, at the date of the inquiry, there were buildings on all sides of the reservoir property, the assumption would lie that like buildings would extend over the whole reservoir site; if buildings were about a portion only of the reservoir property, the higher value of the improved real estate would attach to that proportion of the area of the reservoir which the improved portion bears to the whole perimeter of the reservoir site. Such a method would give to the owner the benefit of the decision of the Federal Supreme Court in the Minnesota rate case, and some part of the increment in the value of the property that is the just due of the owner. Take the illustration given of the New York Central Station. It is equity to assume that the old station was obsolete, and that its original cost had been returned to the owner through earnings during its life, and that a new station of the character provided was necessary for present use and the reasonable requirements of the future. To make a valuation after the cost-of-reproduction method, the assumption is made that all improvements on the station site are obliterated. If the inquiry is made, at the same time, of the cost-of-reproduction of the railways which are using the station, no injustice is done the owner, because there is no traffic to handle. If the inquiry is made as to the cost-of-reproduction of the station only, then the inquiry must make provision for the traffic, either by providing as a part of the cost of the new station other property for temporary use with temporary facilities, or by so augmenting the unit costs of the construction work as to permit the work to be undertaken with all the burdens of handling the traffic existing at the date of the inquiry. The burdens on the work, as a consequence of the proximity of other properties, rights and interests at the date of the inquiry, would rest in either case.

Application of Reproduction Method of Valuation.—The conclusion of the Committee, that "it is necessary to make a deduction from the values obtained * * * for the net accrued depreciation of the physical property due to age and other causes" does not necessarily follow for railway properties. Depreciation is merely an item of maintenance of some part of the plant deferred beyond the period of one year because the nature of that part does not permit the repair to be made within one year. If our accounting were in ten-year periods, instead of one-year periods, many of the items to be provided for as depreciation, and to be deducted from the value of the physical property under the Committee's conclusion, would be considered as items of maintenance, and, therefore, not deducted from the value of the physical property. The difficulty may be overcome by regarding depreciation as a deferred maintenance, to be taken into the annual accounting as a liability of the owner, but not to be deducted from the value of the physical property for rate-making purposes, or for

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any other purpose, any more than any other liability is to be deducted from the value of the physical property, for illustration, the unpaid fuel bills. Being an item of indebtedness of the owner, the accrued depreciation at the end of any fiscal year should be accurately set out in the accounting so that the amount thereof would be a debit against the assets of the owner. The assets of the owner will include items in addition to the physical property of the plant, and it is unjust to the owner to reduce the value of his physical property by a liability necessarily incurred in the operation of the plant, and which his balance sheet shows to be within his ability to meet when the necessity arises, and then on such reduced valuation, to fix the rates to be charged for service, or to fix the amount of a purchase price.

Inventory.—The writer would add, to the Committee's illustrations, the familiar one of the depression of the underlying soil by the weight of railway embankments when constructed over the low-lying lands of the Coastal Plain in the South Atlantic and Gulf States, and on similar soils elsewhere. The amount of this subsidence is not determinable from any measurements, made after completion of the roadway, of the volume of the embankment, although the amount of such subsidence is often 20% and more of the total volume of the embankment.

Unit Prices.—Care must be taken in adapting present-day prices to the cost-of-reproduction method. The present-day prices most naturally chosen will be those current, on the work of the railway under inquiry, for additions and betterments, in connection with like prices on railways in the same general territory. Now, such prices are always based on facilities of transportation afforded by the railway to which the inquiry of cost-of-reproduction is directed, and on other concessions in the way of free transportation, etc. Proper additions should be made to such current prices to ascertain what would be the correct unit prices, were the railway and its branches assumed as being blotted out preparatory to its reproduction under present-day conditions.

Another feature having an enormous effect on unit prices is the demand made at any one time on the plants, organizations, and labor available for railway construction. In order that the assumptions necessary for a cost-of-reproduction may not become absurd, the amount of work proposed to be done, and the time limits set therefor, must be in reasonable accord with the available agencies for construction, all other business being assumed to be normal. This requirement can best be met for the larger systems by assuming that the reproduction work in any one State is undertaken separately from that of any other State, and that the construction agencies of all adjoining States are available. This assumption would fix as a maximum time limit the time necessary to reproduce that portion of the railway system under inquiry which exists in one State only. The suggestion of divi-

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sion into States grows out of the probability that the results of the cost-of-reproduction valuation will be required for use in intra-state discussions as well as in interstate discussions.

The following assumptions may be made in endeavoring to adapt present-day prices in the older parts of the United States to the method of cost-of-reproduction:

1.—That the work will be done under all the present-day conditions; with the available contractors, and with labor of its present amount and efficiency, and with machinery of the several kinds now in daily use; and that the contract of the work will be burdened with all the prejudices, legal hostilities, and greeds held by the public against railway enterprises, which enable the contractors and the public to collect doubtful and unjust claims against the railway company.

2.—That the railway is to be built exactly as it exists at the time the estimate is made (even to repeating all the engineering errors that appear in the existing railway), and that the better railway, in the light of present-day needs, which might be built for the same money is not to be reproduced.

3.—That the railway to be reproduced must be considered as being entirely obliterated; the right of way re-afforested where any growing timber now touches either limit of the right of way, and as swamp land where swamp lands now border the right of way; as fields or meadows when such border the right of way, and as city or town lots for the length the railway exists within the limits of cities and towns. All cuts are to be considered as re-filled, and all embankments and structures of every kind as being entirely removed. These conditions are necessary to enable an estimate to be made of the amount of work to be done.

4.—That with the obliteration of the railway, the facilities of transportation afforded by the railway itself, or its branches no longer are available. There is a definite and determinable relation between the cost of the construction of the roadway, and structures of a railway, and the distances over which material, labor, and supplies must be transported by wagon haul.

5.—That all the facilities of transportation offered by rail lines other than the line to be reproduced, and by all water lines and highways, adjoining the line to be reproduced, are available at the current transportation rates and conditions.

6.—That when a system extending over many States is to be reproduced, it will be advisable to consider the work as being done in certain sections or zones, and to estimate the work of reproduction of each section or zone separately. This assumption is important because it is physically impracticable to construct the whole of an extensive system at one operation, without introducing fabulously high

construction costs, due to the lack of labor and appliances for such construction, when other business is normal in the United States.

7.—That the natural divisions into sections and zones will be by States. In any event, it will be necessary to tabulate the reproduction cost of work by States, and when it becomes necessary to use the reproduction cost in intra-state matters, any item of reproduction cost of the lines within the State that is affected by the assumption of the obliteration of lines without the State cannot be defended in a controversy over intra-state matters.

8.—That within any one State the main lines (both principal and secondary) should first be reproduced, and then the branch lines and spurs. Such an assumption will lessen the construction cost of the branch lines and spurs, because of the transportation facilities afforded by the main lines.

9.—That the time element is of great value, more than the mere calculation of interest on monies expended.

The purpose of these several assumptions is to make clear the conditions to be considered in determining the correct quantities and correct unit costs to be used in the calculations for cost-of-reproduction.

With particular reference to the lines composing the railway systems in the Atlantic Coastal Plain, Fig. 6, the varying character of the soils, native resources, transportation facilities, etc., forbid the adoption of a single unit price for the several items of railway construction, for the system as a whole, or even for all the lines in one State; for example, the dry soils of Florida are generally too soft and too light to permit the economical use of scraper outfits, and the wet soils are too swampy. This condition makes necessary the use of human labor. The dry soils of Southwest Georgia and South Alabama are generally indurated to such an extent as to be too hard for the economical use of scraper outfits, and at the same time, the transportation facilities are too meager to permit the bringing of steam excavating machinery and fuel therefor. This results in high unit costs, due to the use of scraper outfits and human labor on materials not economically adapted for such.

The unit costs for branch lines should generally be taken as less than for the main lines, to be in accord with the eighth assumption, and because the time element is not of such value as in the completion of main lines.

The fixing of unit costs can be arrived at by considering separately each strip of 50 to 100 miles of railway, and fixing unit prices for each strip, which (if there be any desire therefor) may be then averaged for the State as a whole.

As an example, consider the unit cost of grading required for the reproduction of the main line of the Atlantic Coast Line Railroad from the Virginia State line to Wilson, 62 miles. The single-track railway was

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built more than 50 years ago. It has been constantly improved until it is now a double-track main line of the first class, carrying heavy and fast traffic. If the main line and branches are assumed to be obliterated there would remain the following rail and water transportation facilities: Atlantic Coast Line at Virginia State Line, Seaboard Air Line at Weldon, Norfolk-Southern at Wilson, Roanoke River at Weldon, Eastern Carolina from Tarboro south.

The present-day contract unit prices paid by the Coast Line in this territory are confined exclusively to construction of second tracks and other additions to the property. These unit prices were based on the privilege of free transportation of men and materials, etc., over an extensive mileage, and, to adapt them to the cost-of-reproduction method, with no free transportation, and none of any kind during the reproduction period, by that portion of the railway under inquiry, there should be added the following percentages:

1.—The value to the contractor of the free transportation over the Coast Line on employees, tools, machinery, construction material, and fuel for steam shovels, etc., in percentage of the contract price.....10%

2.—The additional cost of construction on account of transporting men and animals, tools, machinery and supplies overland, from the nearest transportation facilities for distances in excess of an average wagon haul of 5 miles; to-wit: 16½ miles at 10% increase for each 5 miles $= 16.5 \times \frac{10}{5} = 33$ per cent. The 33%, however, applies only to the strip from Wilson to Weldon (85% of length Virginia State Line to Wilson), because there are transportation facilities within 5 miles of that portion of the main line between Virginia State line and Weldon, 33% of 85% =.....28%

3.—The additional cost of construction due to the payment of higher wages to labor, necessary to induce labor to go to and remain on the line from Wilson to Weldon, remote from transportation facilities: 20% of 53% of length of line affected = 10.6 per cent. Inasmuch as the labor cost is only about one-half the total construction cost, the total increase in percentage becomes.....5%

4.—The additional construction cost due to increases in prices resulting from the demands made on the labor and material markets of North Carolina were the 553 miles of principal and secondary main line of the Atlantic Coast Line to be reproduced as a single operation: 25% on 50% of total, in percentage.....12%

5.—The additional sum necessary to be procured by the contractor as profits to warrant him in handling the greater amount of capital: 5% of 55%.....3%

Total increase.....58%

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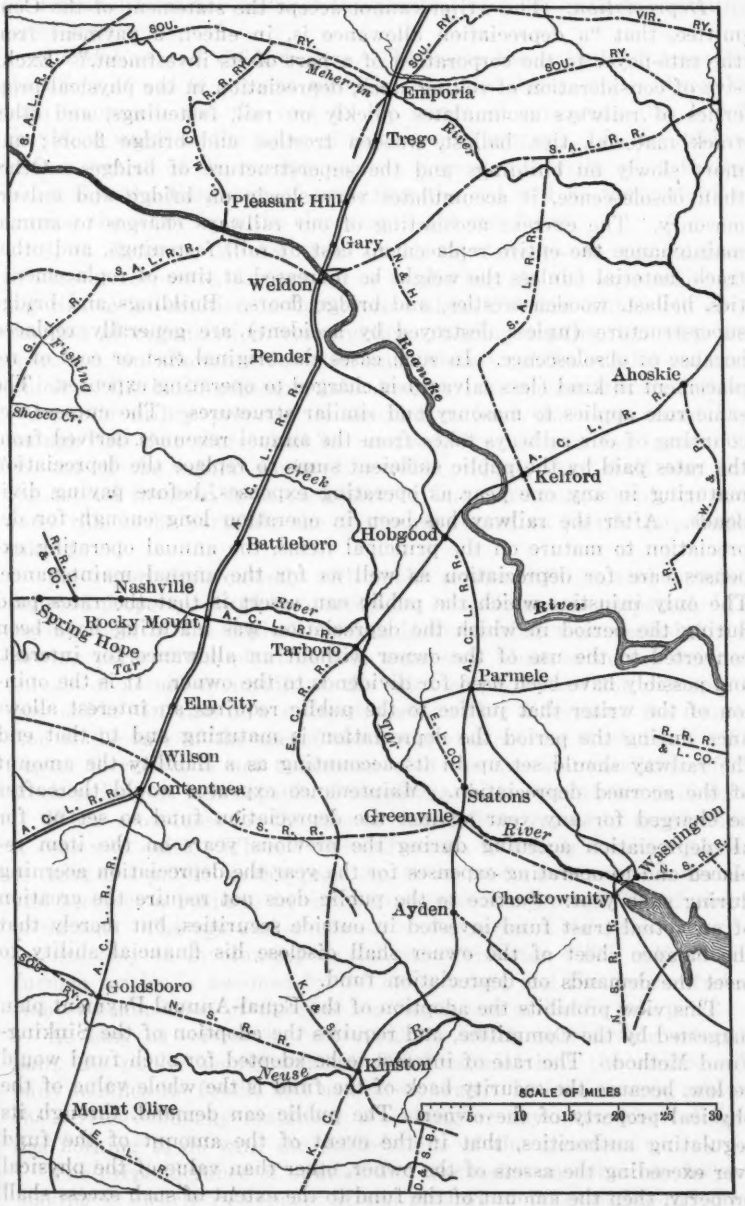

FIG. 6.

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Depreciation.—The writer cannot accept the statement of the Committee, that "a depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment." Exclusive of consideration of rolling stock, depreciation in the physical properties of railways accumulates quickly on rail, fastenings, and other track material, ties, ballast, wooden trestles, and bridge floors; and more slowly on buildings and the superstructure of bridges. Other than obsolescence, it accumulates very slowly on bridge and culvert masonry. The current accounting of our railways charges to annual maintenance the entire replacement cost of rail, fastenings, and other track material (unless the weight be increased at time of replacement) ties, ballast, wooden trestles, and bridge floors. Buildings and bridge superstructure (unless destroyed by accident) are generally replaced because of obsolescence. In such cases the original cost or cost of replacement in kind (less salvage) is charged to operating expenses. The same rule applies to masonry and similar structures. The current accounting of our railways takes from the annual revenues derived from the rates paid by the public sufficient sums to replace the depreciation maturing in any one year as operating expenses, before paying dividends. After the railway has been in operation long enough for depreciation to mature on the principal items, the annual operating expenses care for depreciation as well as for the annual maintenance. The only injustice which the public can assert is that the rates paid during the period in which the depreciation was maturing have been converted to the use of the owner without an allowance for interest, and possibly have been used for dividends to the owner. It is the opinion of the writer that justice to the public requires an interest allowance during the period the depreciation is maturing and to that end the railway should set up in its accounting as a liability the amount of the accrued depreciation. Maintenance expenses would thereafter be charged for any year against the depreciation fund so set up for all depreciation accruing during the previous years on the item replaced and to operating expenses for the year the depreciation accruing during such year. Justice to the public does not require the creation of an actual trust fund invested in outside securities, but merely that the balance sheet of the owner shall disclose his financial ability to meet the demands on depreciation fund.

This view prohibits the adoption of the Equal-Annual-Payment plan suggested by the Committee, and requires the adoption of the Sinking-Fund Method. The rate of interest to be adopted for such fund would be low, because the security back of the fund is the whole value of the physical property of the owner. The public can demand, through its regulating authorities, that in the event of the amount of the fund ever exceeding the assets of the owner, other than value of the physical property, then the amount of the fund to the extent of such excess shall

be deducted from the value of the physical property with the resulting lower service rates. Mr. Wil-loughby

Adaptation and Solidification.—Wherever the growth of grass has the effect of sod to prevent washing from the slopes, the roadway has been improved, and the cost to the owner of procuring this benefit should be added to the value of the physical property. If the roadway be in a section of the country where sod grows naturally, as in Central Kentucky, the total cost of procuring the sod will be the cost of repairing the wash from the slopes until the sod forms.

S. S. ROBERTS, M. AM. SOC. C. E. (by letter).—In any business, private or corporate, for private or public service, the expectancy giving rise to its creation is that there will be a return which will cover the cost of organization, construction, running repairs, renewals, and operation, an allowance for risk, and interest on the capital invested, which is the basic principle laid down by the Committee as follows: "A public utility property is entitled to have the rates made high enough" so that there shall be received, "not only a fair return for the use of the capital invested in such property, but shall also have the capital invested in it paid back." The consensus of opinion indicates this to be certainly the minimum return to justify any undertaking. Mr. Roberts.

The report deals entirely with public service utilities, and especially railroad property. Such properties are in their nature perpetual. The paying back of the capital consists in furnishing that needed for renewing parts of the plant as they wear out. Just as in any business, in sale of actual commodities, the sales supply means for purchase of new stock. In case of a railroad, there is this difference: the transaction is the sale of the use of the stock, and not the stock itself. The buyer, therefore, is not concerned with the age and condition of the stock so long as it is capable of supplying the use or service paid for.

When a business for the sale of commodities ceases to supply stock, it ceases to exist. When a railroad ceases to supply service, it ceases to exist as a public service property, as far as its owners are concerned, and becomes a business in sale of commodities through forced change of owners. The age and condition of the parts of the property are of essence only in involuntary or voluntary transfer. The obligation to furnish service is assumed by the new owners, and the errors of omission or commission of the past are taken care of in the sale price.

As long as service is furnished, for which only a fair return is paid, the public should consider it immaterial how the service company handles its funds, and makes renewals. In other words, whether the service company maintains a sinking or any other kind of fund, has nothing to do with the public's side of the question.

The preponderance of opinion and discussion on valuation for rate-making is clearly in support of the method of "Cost-of-Reproduction-

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New" without depreciation. This places the old and new companies on a more nearly equal basis, and allows the older companies the advantage of the appreciation of their property, which should be their just due in reward for their foresight and risk in undertaking pioneer work.

The writer feels compelled to disagree with the Committee's conclusion as to appreciation. The growth and development of the community and of the railroad serving it are identical. The fact that the property of the community appreciates is due mainly to its advantage of location, which is due to the operation of the railroad to and through it. Their right to enjoy what they have is not questioned. At least, it is not in this consideration of rate-making. The advantage of the railroad is also one of location, whether determined through foresight or chance, which places it in a community capable of development and appreciation through transportation service. Advantage of the appreciation of the community property may be taken only through sale, mortgage, or increased rental. The same is true of railroad property. The appreciation is latent. Depreciation is an operating expense active and dynamic, a renewal of property actually given to the community together with the service rendered; it should be paid for as such, and should not be used to offset appreciation.

Some have argued, in support of the Committee's view, that the right of "eminent domain" is a special privilege to the railroad for which the public should take toll. It is true that very few railroads could be built without this right, but do they not pay full value for all property they acquire under it; and should they not enjoy all the advantages and rights of other property holders who have paid full value for what they possess? Is not the power of eminent domain, as a matter of fact, a question of public policy, so that the individual may not gain at the expense of the many, and is it not created for the public good?

To balance depreciation against appreciation seems to be taking toll twice from the railroad company. Unless the whole system of land values were changed, affecting alike the community and the railroad, is it just to take that from the railroad which is freely allowed to the remainder of the community as its just award?

Fair agreement seems to obtain on all points for which return should be permitted. The most mooted consideration remaining is the manner of providing adequate and, at the same time, equitable means for renewals, so that the return for this purpose may be spread over the period of usefulness of the parts so as to give a total return which shall be reasonably uniform and result in steady, not fluctuating, rates.

It is too trite to mention that he who deals in the future, deals with uncertainty; yet the fact remains that our best decision in regard to provision for renewal funds is an estimate. The one who must bear

the result certainly should be given the advantage of all reasonable doubts. Mr. Roberts.

Alexander C. Humphreys, M. Am. Soc. C. E., in his paper, "Depreciation: Estimated and Actual," shows the error of considering the average life of a plant as a whole in making provision for depreciation, or, as the writer prefers to express it, renewals. Mr. Humphreys emphasizes the necessity of considering separately each group of parts having the same period of usefulness. He and others give warning that, in order to hope for results sufficiently near to the probable actual performance of any part to warrant consideration, careful account must be taken of the past history of the part and of its present condition, as determined by actual examination.

Considering that the period of usefulness of a part has been determined as accurately as human knowledge and foresight will permit, now comes the problem of providing a means to secure its perpetuation.

The methods of computation form the academic considerations of the problem; the equity lies in the adoption of that method which gives just results under all conditions.

There is agreement between all writers that the Sinking-Fund Method of computation, considering return on the "Cost-of-Reproduction-New" without depreciation, is correct in ultimate result under all assumptions of rate of interest for return on invested capital and for computation of the annual increment for depreciation or renewal. It makes no difference whether the rate of interest for return on invested capital is the same or different from that used in computing the annual sinking fund or renewal increment.

It is generally conceded, too, that such a basis of computation represents a minimum permissible return, because the annual increment, must, on its receipt, be at once put out at compound interest in order to equal the necessary accumulation during the period of usefulness of the part.

Where the rate of interest allowed for return on invested capital and for computation of the annual depreciation or renewal increment is the same, the ultimate result of the Sinking-Fund Method and of the Equal-Annual-Payment Method is identical. This is shown in Table 16.

C. E. Grunsky, M. Am. Soc. C. E., in his paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates",* in "Table 8, Method 1", and "Table 8, Method 3", and also in his Fig. 3, and in his explanation of the different methods, confirms Table 16.

It is evident that where the rate of interest is the same, there is a distinction, but not a difference, in these two methods, and any difference in distribution of the total annual return is imaginary.

* *Transactions*, Am. Soc. C. E., Vol. LXXV.

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TABLE 16.—PERIOD OF USEFULNESS, 20 YEARS; RATE OF INTEREST, 5 PER CENT.

Age, in years.	SINKING-FUND METHOD.			EQUAL-ANNUAL-PAYMENT METHOD.			Ultimate or total return, either method.
	Capital invested.	Annual renewal increment.	Annual return on capital invested.	Depreciated value of capital invested.	Annual depreciation or renewal increment.	Return on depreciated capital.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	$100.00	.....	.....	$100.00	.....	.....	.....
1	100.00	$3.02	$5.00	96.98	$3.02	$5.00	$8.02
2	100.00	3.02	5.00	93.80	3.18	4.84	8.02
3	100.00	3.02	5.00	90.47	3.33	4.69	8.02
4	100.00	3.02	5.00	86.97	3.50	4.52	8.02
5	100.00	3.02	5.00	83.29	3.68	4.34	8.02
6	100.00	3.02	5.00	79.43	3.86	4.16	8.02
7	100.00	3.02	5.00	75.38	4.05	3.97	8.02
8	100.00	3.02	5.00	71.12	4.26	3.76	8.02
9	100.00	3.02	5.00	66.65	4.47	3.55	8.02
10	100.00	3.02	5.00	61.96	4.69	3.33	8.02
11	100.00	3.02	5.00	57.03	4.93	3.09	8.02
12	100.00	3.02	5.00	51.86	5.17	2.85	8.02
13	100.00	3.02	5.00	46.43	5.43	2.59	8.02
14	100.00	3.02	5.00	40.73	5.70	2.32	8.02
15	100.00	3.02	5.00	34.74	5.99	2.03	8.02
16	100.00	3.02	5.00	28.45	6.29	1.73	8.02
17	100.00	3.02	5.00	21.85	6.60	1.42	8.02
18	100.00	3.02	5.00	14.92	6.93	1.09	8.02
19	100.00	3.02	5.00	7.64	7.28	0.74	8.02
20	100.00	3.02	5.00	0.00	7.64	0.38	8.02

The Equal-Annual-Payment Method, as compared with the Sinking-Fund Method, the basic rate of interest being the same, is simply the shifting of a part of the contents of one of two pockets to the other, without changing the total contents of the pockets taken together.

The foregoing comparison ceases to be true when, for any reason, the rate of return on the capital invested is different from that used in computing the depreciation or renewal increment:

(1) When the rate of return on the invested capital is greater than that allowed in computing the renewal increment, then the Equal-Annual-Payment Method ceases to be an Equal-Annual-Payment Method in any sense of the word, and will work a continually increasing injustice on the public service company after the end of the first year until the end of the period of usefulness of the part under consideration. This is shown by Table 17.

(2) When the rate of return on the invested capital is less than that allowed in computing the renewal increment, then the Equal-Annual-Payment Method again ceases to be an equal-payment method in any sense of the word, and will work a continually increasing injustice on the rate-payers after the end of the first year until the end of the period of usefulness of the part under consideration.

TABLE 17.—PERIOD OF USEFULNESS, 20 YEARS; RATE OF RETURN ON CAPITAL INVESTED, 7%; RENEWAL RATE, 5 PER CENT. Mr. Roberts.

	SINKING-FUND METHOD.				EQUAL-ANNUAL-PAYMENT METHOD.			
Age, in years.	Capital invested.	Annual renewal increment.	Annual return on capital invested.	Ultimate or total return.	Depreciated value of capital invested.	Annual depreciation or renewal increment.	Return on depreciated capital.	Ultimate or total return.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	$100.00	.....	.....	.....	$100.00	.....	.....	.....
1	100.00	$3.02	$7.00	$10.02	96.98	$3.02	$7.00	$10.02
2	100.00	3.02	7.00	10.02	93.80	3.18	6.79	9.97
3	100.00	3.02	7.00	10.02	90.47	3.33	6.57	9.90
4	100.00	3.02	7.00	10.02	86.97	3.50	6.33	9.93
5	100.00	3.02	7.00	10.02	83.29	3.68	6.08	9.76
6	100.00	3.02	7.00	10.02	79.43	3.86	5.83	9.69
7	100.00	3.02	7.00	10.02	75.38	4.05	5.56	9.61
8	100.00	3.02	7.00	10.02	71.12	4.26	5.27	9.53
9	100.00	3.02	7.00	10.02	66.65	4.47	4.98	9.45
10	100.00	3.02	7.00	10.02	61.96	4.69	4.67	9.36
11	100.00	3.02	7.00	10.02	57.03	4.93	4.33	9.26
12	100.00	3.02	7.00	10.02	51.86	5.17	3.99	9.16
13	100.00	3.02	7.00	10.02	46.43	5.43	3.63	9.06
14	100.00	3.02	7.00	10.02	40.73	5.70	3.25	8.95
15	100.00	3.02	7.00	10.02	34.74	5.99	2.85	8.84
16	100.00	3.02	7.00	10.02	28.45	6.29	2.43	8.72
17	100.00	3.02	7.00	10.02	21.85	6.60	1.99	8.59
18	100.00	3.02	7.00	10.02	14.92	6.93	1.53	8.46
19	100.00	3.02	7.00	10.02	7.64	7.28	1.04	8.32
20	100.00	3.02	7.00	10.02	0.00	7.64	0.53	8.17

NOTE.—In Tables 16 and 17, the computations of the Committee are used.

A table will not be given to illustrate this. Such a premise can hardly be imagined to obtain in practice. It is sufficient to state that, considering a rate of return on capital at 5%, and computing the replacement increment at 7% for a period of usefulness of 20 years, the ultimate or total return at the end of the first and the twentieth years will be:

	Sinking-Fund Method.	Equal-Annual-Payment Method.
1st year.....	$7.44	$7.44
20th "	7.44	9.26

The Sinking-Fund Method and the Equal-Annual-Payment Method are thoroughly discussed by Mr. Grunsky in the paper, previously referred to, as "Method 1" and "Method 3", respectively, and their relative advantages and disadvantages are described by him as follows:

"Method No. 1 [Sinking-Fund Method] is always applicable when it can be shown that earnings have been adequate in the past, no matter whether the property is a single item, or is composed of many items; whether the expectancy is long or short; whether the expectancy is uniform for all parts of the property or not; whether the plant is of mature age or not; whether the property has attained full growth

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or whether it is still growing at a uniform rate, or otherwise. It is simple of application, and does not involve determination of the present condition of the property, provided that it is maintained in proper condition to render adequate service. It furnishes all the information necessary for intelligent action in fixing rates, because when it is known what the net earnings, above operating expenses, must be to yield the return which money should earn in ordinary safe investments, then an arbitrary addition can be made, to compensate the owner for management and risk of loss.

"In contrast with these advantages, Method No. 3 [Equal-Annual-Payment Method], under which value as ordinarily understood must be determined by deducting depreciation from the investment, requires a special determination of value for each item of which the property is composed, and a new determination every year for every item, or in special cases, for every group of items of the same expectancy. Each item has a new value each year and a remaining life which grows continually shorter. Amortization, therefore, must be estimated on a new basis each year. The judgment of the expert is called into play to determine the condition and probable remaining life of the several parts of the property, and after the complex calculation is made, if the same basic rate of interest is used throughout, the result should agree absolutely with the simpler Method No. 1."

No greater part of the investment is returned to the investor by the Equal-Annual-Payment Method than by the Sinking-Fund Method. The Equal-Annual-Payment Method is more difficult to compute, and gives correct results only under the condition of equal basic rates of interest.

If Table 16 is correct, then in Table 6,* of the Committee's report it is evident that part of the return on the capital invested is shown reinvested in additional plant as "earnings for depreciation". Also, at the end of the first year, after the depreciation or replacement increment of $7 238 is reinvested in additional plant, the plant is furnishing all the service any $107 238 plant could possibly furnish. The original $100 000 investment and the $7 238 investment, as far as the plant and service are concerned, and as far as ultimate return is concerned, should be treated as entirely separate and distinct investments, each of which enjoys in every respect the same and all rights and privileges accorded to capital to earn.

The illustrations of notes and their payment used by the Committee do not appear to be in all cases *apropos,* for the reason that the perpetual or continuous nature of the property is not considered. That is, that something in railroad properties, termed "Latent Depreciation", by Richard J. McCarty, M. Am. Soc. C. E., in his paper, "Depreciation of Railroad Property", is not taken into account.

The adoption of the term "depreciation fund" seems, in a way, extremely unfortunate, and to it may be attributed much of the mis-

* "Addition to the Progress Report of the Special Committee."

understanding of the purpose for which it is created; especially in the minds of those not thoroughly familiar with the whole subject. Although, of course, this notation arises from the fact that the basis of computation of the fund is wear and tear, and final exhaustion, still the fund is not for depreciation, but for continuation and perpetuation, and is, it seems, more happily expressed as a "Renewal Fund", which certainly is its purpose. The rate-payers get the whole of the plant, along with their service. The "Renewal Increment" is proportional to the rate at which the plant is used up by them. The "Renewal Fund" is something due the service company, not something to be taken from it. Mr. Roberts.

The writer believes the "Sinking-Fund" or "Renewal-Fund" Method to be generally more equitable than the proposed "Equal-Annual-Payment" Method.

Considering that it requires all the "Renewal Increments" increased by all their interest accumulation to equal the "Renewal Fund"; considering the uncertainty of forecasting future performance; considering the advance of the art which frequently requires the renewal of parts before their expected period of usefulness can be enjoyed; considering the mature age of the majority of railroad properties, which have reached that stage where renewals are uniform and constant; that is, considering the "risk", liberality should be used in dealing with the allowance for and make up of the "Renewal Fund".

C. P. HOWARD,* M. AM. SOC. C. E. (by letter).—The "Sinking-Fund" Method provides: Mr. Howard.

(1).—Equal annual payments, which at compound interest will amount to the full value of the plant at the end of its life.

(2).—Return on capital computed for the full value of property.

The owner then receives a return (say, 7%), on the full value of his investment throughout its life; and the sinking-fund increments put out at compound interest (say, 5%), will return him the full value of the plant at the end of its life. If these increments, computed on the basis of 5% interest, were by good management invested at 7%, the owner would have a surplus of more than enough to renew his plant.

The "Equal-Annual-Payment" Method provides for the repayment of the invested capital. Mr. Roberts points out that this gives a considerably less return to the investor than the Sinking-Fund Method, when depreciation allowances are calculated at a less rate of interest than return on capital.

The reason for this is obvious. If the owner could re-invest immediately each depreciation allowance in the plant, or in other business, which would earn 7%, without any loss of time, or development period, he would at all times have 100% capital invested with annual returns

* Previous discussion by Mr. Howard appeared in *Proceedings* for February, 1914, on p. 387.

Mr. Howard. of 7%, as indicated in Table 6, of the Progress Report. Of course, he could do better than this with the larger payments received under the Sinking-Fund Method; but the very reason for computing sinking-fund increments at a lower rate of interest is the supposition that capital must be kept on hand and available for more or less irregular expenditures, as they occur. It must be ready cash. If it is re-invested, the company will have to go to the expense of borrowing on short-term notes or some other method of securing ready cash.

Referring again to Table 6, and what it implies: Is it reasonable to suppose that the company can always immediately re-invest every dollar in improvements and extensions? Except to a limited extent, it is evident that improvements and extensions cannot be planned and financed in such manner; and, if they could, it would be idle to expect full earnings from the start. It is possible, of course, that a part of the fund might be re-invested from time to time at 7%, the rest remaining on deposit at a lower rate of interest, say 3 per cent. The average interest return on the sinking fund would then be somewhere between—say, 5 per cent.

The matter could be handled easily enough under the Sinking-Fund Method, whatever disposition is made of the fund. The only thing to decide is what is the reasonable average rate of interest at which the sinking fund may be expected to accumulate when invested under ordinary conditions with reasonably good judgment.

Mr. Mortimer. JAMES D. MORTIMER, ESQ. (by letter).—Judging by the title, the Committee has confined its preliminary report to valuation for the purpose of rate-making. There are other purposes for which a valuation may be made, such as purchase and sale, taxation, municipal or governmental acquisition, etc., which would introduce other elements of value.

As the report is preliminary and the discussion now called for partakes of a like nature, the writer's remarks are purposely brief, and designed to sketch the line of investigation of statistical and economic principles now being conducted by several public utilities and the Bureau of Fare Research of the American Electric Railway Association.

1.—The process of assembling the elements of value is very much simplified by concentrating the attention on the property of the utility, as distinguished from that of the corporation owning it. The utility is the entity possessing the franchise or permit to occupy the streets, the physical property used or useful in the performance of the public service, and the other assets necessary to the operation of the complete plant. This conception is much simpler than that of principal and agent, which some have recently endeavored to introduce for the purpose of defining the relation between the State, the public, and the public utility corporation.

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2.—The determination of the valuation for the purpose of rate-making consists of assembling the elements of value or assets of the utility. These assets consist not only of the physical property used or useful, but also all other assets, such as accounts receivable, which the utility may have owing to it by the corporation. Many of the reserve liabilities of the corporation, such as depreciation reserve, fire insurance reserve, injuries and damages reserve, are, in the proper accounting conception, assets of the utility and entitled to consideration in determining its value.

3.—In dealing with the subject of depreciation it is of importance that the various conceptions be recognized and distinguished, *viz.:*

(*a*) Rate at which depreciation is accruing at the moment or during the current year;

(*b*) Rate at which depreciation has accrued during the previous life of the unit of plant;

(*c*) Rate at which depreciation will accrue during the remaining life of the unit of plant;

(*d*) Annual reserve required to finance replacements as they occur from time to time;

(*e*) Accumulated depreciation measured by loss in value or utility;

(*f*) Accumulated depreciation reserve necessary to finance replacements as they occur from time to time.

No satisfactory definition of accrued depreciation or depreciated value has yet been developed which does not bear some relation to the estimated amount of money which would have accumulated under some plan of annual payments commencing at the time the unit of plant was placed in operation. Whether the problem of measuring accrued depreciation scientifically, by either inspection or statistics, will be separated from that of financing replacements, cannot now be predicted with any certainty.

4.—The actual future life of any particular unit of plant ascertained by inspection is indeterminate. Only the boundary conditions are questions of fact; the unit may be either new or about to be abandoned. With respect to all intermediate conditions, the facts are indeterminate; abandonment may occur to-morrow or 10 years hence.

5.—Experience and general observation show that there are usually two concurrent forces acting to limit life, that is, to require replacement or abandonment. These two forces are probably present in different relative proportions in each class of equipment, and it may be that in many classes either one or the other may be almost entirely absent. These two forces are defined as follows:

(*A*) The force causing destruction, remaining constant throughout the life of the unit, acting equally on new and old, and covering destruction arising from chance, such as accident,

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governmental requirement, inadequacy, improper design, and engineering errors in design, selection, and construction.

(*B*) The force causing destruction, increasing directly with age, acting on the old in greater proportion than on the new, and covering destruction arising from wearing out or other causes which vary directly with age, such as obsolescence and natural decay.

The relative proportion in which these two forces are present in any class of equipment cannot be determined until after the causes of depreciation have been quantitively measured in the light of a proper gathering and assembling of actual data.

If L = proportion of original number living at age, t; then,

$L = e^{-At}$, where only the force, A, is present;

$L = e^{\frac{-Bt^2}{2}}$, where only the force, B, is present; and,

$L = e^{-t\left(A+\frac{Bt}{2}\right)}$, where both forces, A and B, are present.

Where e is the base of the natural system of logarithms.

6.—The problems involved in the determination of the proper amount to be included in operating expenses, to insure the replacement of the various plant units, are similar to those involved in the calculation of life insurance premiums. The amount, per dollar invested in a particular plant unit, required for such property replacement insurance depends on the age of the unit, the mortality table of a group of such units, the reserve already accumulated, the rate of interest allowed on accumulations in the reserve, and the evident state of preservation of the unit. The accumulations in reserve may be drawn on at any time, depending on the contingencies requiring the abandonment of the unit of plant. As the maturity dates of the loans making up the depreciation reserve of the utility are indeterminate, and may be one day or several years, the loan is entitled to a rate of interest comparable with that allowed on daily bank balances or the rate of interest assumed in calculating life insurance premiums, say, 3 per cent. This interest rate should be distinguished from that assumed for depreciation calculations, where an average life is estimated and the deficiencies in interest and reserve accumulations arising from early mortality are assumed to be offset by the greater interest accumulations during years beyond the assumed mean life. If the replacements required in a typical complete plant be set down in detail during a period of time running beyond the average life of the plant, it is found that the allowable interest rate resulting from interest accumulations is very low, approximating 2% in the case of some plants having an estimated average life in excess of 30 years.

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7.—The rate of interest allowed on depreciation reserve accumulations is a very different quantity from the rate of return to be allowed on permanent investment in plant; the former may be withdrawn in large part at any time, but the investment in operating plant must remain until worn out or otherwise abandoned. It is also important at this place in the discussion to recognize the fact that public utilities as viewed in law are presumed to operate in perpetuity, and the owners of a utility cannot withdraw the capital invested in any plant unit when the depreciation reserve has returned an amount equivalent to the original investment. These owners, by virtue of the public functions assumed on behalf of the utility, are required to continue the plant in operation and make the needed replacement. In view of the obligations assumed by the utility, it is unreasonable to assume that accumulations in the depreciation reserve, created, as they may be, from payments by the users of the service, are in the nature of repayment to the owners of the utility of their capital investment. Only in the case where such depreciation reserve accumulations were distributed in the form of interest and dividends on securities of the corporation can it be said that there has been a withdrawal of the invested capital.

8.—From the nature of the property making up a public utility, the problems of valuation and rate-making should be the same, whether the utility be owned by a municipality, the State, an individual, a partnership, association, or corporation. Regulation has primarily to do with the functions of the utility. In the broad sense, the law has no interest in the nature of ownership of the utility, except to the extent that it prescribes and limits the powers of the particular entity in which such ownership may be vested. If these statements be true, it follows that capitalization, dividend record, and ratio between outstanding bonds and outstanding share capital, have no direct relation to the regulation of rates of a public utility owned by a corporation. Likewise, the rate-payers have no interest in stock dividends or book surplus appearing on the books of the corporation.

9.—These considerations permit one to draw the distinction existing between the "going cost" of the utility and the "going value" of the corporation, shown in the latter case by the excess of market value of securities over the cash cost of property acquired thereby.

10.—Specific comments might be made on various parts of the Committee's report, but it is doubtful whether space and time available for the present discussion will permit. Summarized, the principal defects in the Progress Report of the Committee are as follows:

(*a*) An erroneous conception of the function of depreciation allowances is adopted. Depreciation allowances are not repayment of capital unless actually disbursed as interest

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and dividends, and then only when and to the extent that the utility has earned a reasonable return.

(*b*) Depreciation reserve credits appearing on the books of the corporation are among the working assets of the utility, and hence additive to the reproduction cost for the purpose of determining valuation for rate-making; and, if any utility and its corporation have reserved 100% of the estimated accrued depreciation reserve, there results a sum equal to the reproduction cost new.

(*c*) Permanent investments in operating plant, of depreciation reserve accumulations, are entitled to the same rate of return as are investments of the proceeds of bonds and share capital.

(*d*) The very large probable errors contained in all present-day estimates of depreciation allowances, arising from lack of data covering the useful lives in service of various units of plant, do not justify extended discussion as to the relative merits and equity of the "straight-line", "sinking-fund", and "equal-annual-payment" methods. When the principles of actuarial science are properly applied to the problems of depreciation, much minute discussion of unimportant arithmetical methods will be avoided and some definite progress will be achieved.

(*e*) Substantial revision in the economic theories and mathematical principles of the Committee is required before its report can receive general adoption by engineers, public utility operators, regulating commissions, and the Courts.

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CHARLES S. CHURCHILL, M. AM. SOC. C. E. (by letter).—After carefully studying the report of the Special Committee, and after reading several published discussions thereon and listening to an explanation by the Chairman, Mr. Stearns, followed by several verbal discussions, the writer is greatly disturbed by the differences in interpretation of its meaning and the consequent dangers that may result in case it is accepted as the views of the Society on this particular subject.

The writer is particularly impressed with the thought that the questions raised therein are as much legal as engineering in character, or more so, and while not criticizing the Committee for taking a different view, he nevertheless felt himself not altogether competent to pass judgment on these legal questions. He, therefore, submitted the report to a legal friend for his opinion. In much of what follows the writer is indebted for suggestions to this friend, who, though he has no desire to take part in the discussion, has no objection to this use of his opinions.

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In its opening statement the report advises that the Committee preparing it was appointed in September, 1911 (which was prior to the enactment of the Federal Valuation Law), and that it was "to formulate principles and methods for the valuation of railroad property and other public utilities", but that it has submitted "a report covering the subject of Valuation for the Purpose of Rate-Making". Thus, what the Committee was to do, according to its own statement, and what it has done, as appears from its report, do not agree.

Subsequent to the appointment of the Committee, Congress enacted a Valuation Law applicable to "all property owned or used by every common carrier" subject to the provisions of the "Act to Regulate Commerce", which law became effective by the approval of the President on March 1st, 1913. The report of the Committee is made as of December 1st, 1913, for submission to the Society at its Annual Meeting on January 21st, 1914, and yet this Federal Law, which is absolutely controlling in the valuation of railroad properties, and had been in force for 9 months prior to the date of the report, is not referred to in that report, nor was the report prepared with any reference thereto, as is shown by its contents.

The purpose of the Federal Valuation Law is to obtain the value of the various railroad properties of the United States. This law in its opening statement provides that the Commission "shall, as hereinafter provided, investigate, ascertain, and report the value of all the property owned or used by every common carrier subject to the provisions of this Act". Further, it provides that "such investigation", by the Commission, "shall show the value of the property of every common carrier as a whole, and separately the value of its property in each of the several States and Territories and the District of Columbia, classified and in detail as herein required"; it provides that "All final valuations by the Commission and the classification thereof shall be published and shall be *prima facie* evidence of the value of the property in all proceedings under the Act to Regulate Commerce, as of the date of the fixing thereof and in all judicial proceedings, etc."; it provides that the Commission, in addition to investigating, ascertaining, and reporting "original cost to date", "cost of reproduction new", and "cost of reproduction less depreciation", shall, in like manner, "ascertain and report separately other values and elements of value, if any, of the property of such common carriers".

Judge Prouty, Director of Valuation under this law, in a speech on February 11th, 1914, before the United States Chamber of Commerce, said "Up to the present time the holding of the Supreme Court of the United States is, that the cost of reproduction new, or cost of reproduction less depreciation, are only factors entering into the final question of value. Many other things have been enumerated by that Court as bearing upon the value of the property". Again, in the same

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speech, he said "I simply call your attention to the fact that the Commission is required, not merely to ascertain the cost of reproduction, but to state the value of the property, and that in attempting to do so many delicate and difficult questions may be encountered".

The law, thus ascertained, clearly directs that "the value" of the property shall be ascertained. The purpose of this value, whether for the regulation of the issuance of securities, for the protection of investors, as suggested by Judge Prouty, for making rates or for testing or limiting them for taxation, or for some other purposes, is not stated. The valuation may serve many purposes, but whatever the purpose, it is "the value" of the property that must be considered. The law requires "the value" to be reported. Thus it reads and thus has Judge Prouty publicly construed it. No reason, therefore, exists and no purpose is now to be served, when the great work of ascertaining "the value" of "all the property" of every common carrier subject to the law is going on, for discussion, consideration, or conclusions on a particular kind of value, namely, a value for rate-making purposes, because such special values as this are not now under consideration by the Government and are not involved under the Valuation Law.

In the Summary (page 70 of the report), Paragraph 1 reads:

"This report is limited to the discussion of valuation for the purpose of rate-making. Valuations for other purposes differ in some respects, for legal and other reasons".

Also, on page 1 of the report, the Committee says:

"Valuations of the property of public service corporations may be made for a variety of purposes, the most important of which are the furnishing of a basis for the regulation of rates, and for taxation, capitalization and purchase".

It is manifest from this language and from a reading of the entire report that the Committee has been more concerned with a certain "purpose" of valuation, namely, rate-making, than with valuation itself. The Committee proposes methods for arriving, not at "the value" of the property, but at a sum or an amount of money, which, while it will not be the value of the property, will be the value on which the Committee recommends the carrier should be permitted to earn.

The Committee is not happy in the language it uses, as a justification for its course, namely:

"It is sometimes held that valuations for all these purposes [regulation of rates, taxation, capitalization and purchase] should give identical results. This view cannot prevail if the laws of the different States and certain decisions of the Courts are complied with. For instance, there are many State laws indicating how the property of public service corporations shall be valued for taxation and these laws differ radically in their requirements. There are other reasons why the valuation for different purposes should differ, but this feature will not be

discussed at present as this report will, as stated, relate to valuation for the purpose of rate-making". Mr. Churchill.

The omission of these "other reasons" and of any reason whatever for the statement that value for capitalization, for purchase, and for rate-making, is not the same, detracts greatly from the report. The mere fact that one State may tax public service corporations on income, another State by license, and another by *ad valorem*; or that one State using the *ad valorem* method may assess the property at one percentage of its value and another State, using the same method of taxation, may tax at a different percentage of value, illustrates nothing as to value. One State prefers one method of taxation and another State a different method. The fact that different States use different methods of taxation is not even suggestive that the measure of "the value" of the property varies in the different States.

To assume, as the Committee evidently has done, that the American Society of Civil Engineers has instructed it to report on the valuation of railroads "for rate-making purposes", or to assume that the Society will sanction the report though unauthorized, is, in each case, scarcely authorized. In the first case the directions to the Committee are in opposition to the assumption, and, in the second case, the long-established reputation of the Society for the broad treatment of all subjects coming before it equally opposes the presumption there indulged. When has any State ever made a tariff of rates or an individual rate for any railroad based on the value of the railroad's property? What State has ever proposed to do so? How would it be possible for a State or for the Federal Government to do so? A railroad may invoke the value of its property, as has been done, to show that an entire body of rates is confiscatory. In such case, however, it is not some peculiar kind of special value that is invoked, but "the value" of the property. The Constitution does not recognize fantastic differences over supposed different kinds of values, but it protects "the value" of all property, whether that of a common carrier or of some one else. It may be desirable for the Government—State or Federal—to be possessed of official information as to "the value" of a given carrier's property, so that, if a charge of confiscation through rates which are too low is made, the Government will be armed with the facts and will not be dependent on the evidence of value furnished by the carrier, and such, perhaps, is the purpose, or one of the purposes, of the Valuation Law. Manifestly, if such be the purpose, the only value that will be worth anything to the Government—State or Federal—if the charge of confiscation under the Constitution be brought against it, is "the value" of the property. Any figure admitted to be less than the true value of all the property of the carrier, will be worthless, as long as the Constitution stands, and, therefore, labor and cost now expended to obtain such a figure are time and money thrown away.

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There may be a relation between the total net earnings of a common carrier and the total value of all the property of that carrier, speaking always of value as actual value, but there is not, nor can there be, any relation between the total value of all the property of a carrier and one of the individual rates of that carrier. The carrier may be earning a total that furnishes unusually large dividends and yet the particular rate involved may be unduly or unreasonably low, or necessarily fixed by competitive or other conditions.

On the other hand, a carrier may not be earning any dividends, it may not be paying all its fixed charges, and yet the particular rate at issue may be unreasonably high. The reasonableness of the individual rate must be determined in each instance on other considerations than the value of the carrier's property. Under the law each individual rate must be "reasonable", and whether the roadbed of a carrier cost $10 000 or $200 000 per mile to construct, and whether the carrier claims or proves its property to be worth one sum or another, it can never exact under the law any more than a reasonable charge for each particular service that it performs. Thus is the public protected against supposed or actually inflated values, if made or claimed by the carrier. On the other hand, the carrier is entitled, under the law, to make a reasonable charge for each particular service that it renders, so that, regardless of any theoretical, imagined, or depressed value that may be put on its property, it may continue to demand a reasonable charge for each of its services, and thus it is protected. It is true that the term "reasonable" is an elastic one, and throughout all the years in which various commissions have been applying it, no uniform rule for its application has ever been established. This does not mean, however, that it has not been applied, nor that eventually more general rules for its application will not be evolved. Heretofore, each individual rate case has been determined on its own peculiar facts, and such will be the situation after the carriers have all been valued and until such time as uniform rules may be worked out. When worked out, and in the process of working them out, it will be found that, save in the case of a body of rates attacked as confiscatory, the value of the carrier's property will probably cut no figure; but whether or not this be true, it will certainly be found true, as long as the present Constitution exists, that in the making or unmaking of railroad rates—either as a whole or individually—by Government decree, the only value of the property of the carrier that can be considered is its actual value, or, in other words, "the value" authorized and directed by the law to be obtained.

Judge Prouty, in his speech before the United States Chamber of Commerce, to which reference has already been made, used this language:

"The rates of public utilities are at the present day usually fixed by Commissions, both State and Federal. It is perhaps the natural

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inference that when the value of the property has been determined and the rate or return fixed, the work of the Commission in establishing the charge of the public utility is comparatively easy. It is only necessary to multiply the value by the rate and to allow a charge which will yield that income. And this, with some important qualifications, is true as to certain kinds of public utilities. Take, for instance, a water plant or a gas plant. This serves a single community. As a rule it meets no competition in that service. The amount of service is known or can be forecast with reasonable accuracy. Even matters of depreciation and such like have come to be pretty actually understood. It is possible, therefore, to fix with some confidence the rates of such a utility, when the value of the investment is known. With the railroad, however, this is entirely different for the reason that it seldom happens that a single railroad can be considered by itself. The greater part of the business of the railways of the United States is subject to competitive conditions of one sort and another, which are largely controlling so that the rates of one are necessarily bound up with those of another".

Also:

"The railroads of this country are so bound up together that their rates are largely interdependent. It is impossible to shake a single railroad free from every other and fix its charges upon the basis of a fair return upon its fair value as you would in case of a gas or water plant."

Also:

"While, however, I wish to make it perfectly plain that the problem of establishing railroad rates will not be solved by this valuation, I desire to say with even greater emphasis that the problem will be enormously simplified. It can be known with certainty whether the general level of rates is or is not too high and in establishing the charges to be observed by a single carrier, even in fixing the rate upon a single commodity, it will be of much benefit to know the value of the property involved".

All of which means that, in the opinion of Judge Prouty, and as any serious consideration of the subject will show, there is not in law or reason any such thing as a special value different from real value on which to base railroad rate-making, and that the only value of use or service in any degree in the testing or making of rates is "the value" of the property. Judge Prouty, in making rates for a gas or water plant, would want to know "the value of the investment" and not some fanciful and untrue value based on a theory. In the case of railroads, as we have heretofore seen, he construes the law to require "the value" of the property to be ascertained, and now we see that when "the value" has been ascertained, he cannot solve the problem of rate-making on that value. The most he can say is that, knowing "the value", he can know with certainty whether the general level of rates is too high or too low, and that in other respects it will be of much benefit to know "the value" of the property. What this benefit will

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be he does not say, but, accepting his statement that there will be a benefit, it is important to note that the benefit will come from knowing "the value" of the property. The problem of railroad rate-making is an involved and complicated one. Many and conflicting elements enter into it. With such a problem to solve, it should require no argument to demonstrate that injecting into it a fanciful and untrue value of the property involved, not only adds nothing to the solution of the problem, but actually destroys any test of whether the general level of rates is too high or too low.

On page 9, the Committee states that in examining accounts to secure costs of a property that is not old, "it will be necessary to scrutinize the accounts and to examine the property to ascertain whether the construction was carried out in a judicious manner. * * * whether the cost was reasonable".

The writer submits the opinion that, without living witnesses, it is doubtful whether disapproval of properly kept accounts would or should stand when made by any outside party not personally acquainted with the property and its history.

Page 10 of the report is evidently written with the erroneous idea that maintenance, replacements, depreciation, and obsolescence of a railroad, say, 50 years old, has not been generally covered by charges to operating expenses or from income; and, further, under the assumption that on ordinary railroad properties care has not been usually taken to charge to capital account not more than the true betterment or addition. The Committee is in error in making such a sweeping assumption the basis of one of its important conclusions adverse to the records of railroads.

The law directs the Commission to report "original cost to date" of all the property of each carrier, and the duty of the Commission to ascertain and report the figures representing such cost is mandatory; notwithstanding this fact, however, the Committee, instead of offering helpful suggestions for the discharge of the duty, ignores the requirements of the law and on page 11 of its report says: "For reasons given, the Committee believes a valuation of an old property based on its actual cost to be generally impracticable, making it necessary to adopt the cost-of-reproduction method".

On page 15 of the report, the Committee says:

"In view of these and other considerations, the Committee is of the opinion that valuation should be based upon the physical conditions existing at the time the various portions of the property were built, but at the prices prevailing at or near the time of the valuation".

It is to be observed (*a*) that the Committee, ignoring the law under which railroads must be valued, gives no consideration to "original cost to date", which the Commission must ascertain and which will show what it actually cost to construct the property under the condi-

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tions existing at the time of construction; and it is also to be observed (*b*) that the Committee is clearly influenced in its suggestion by the "purpose" for which it would get a certain kind of value; for certainly it would seem that if the physical value of a certain property is the cost of reproducing that property, then the cost of reproduction to be ascertained is the cost to-day, for it is the value of the property to-day —at the time it is being used—and not a value existing years ago, to which the protection of the Constitution extends. For all purposes of value it is the value "to-day" which is involved, and which must be shown.

On page 18 the report reads:

"Although the Committee believes that in such cases as those cited the property to be valued is that which the corporation has paid for rather than that to which it holds title, there are obviously many cases in which property donated to the corporation should be included in the valuation."

On page 50, it reads:

"Your Committee is aware that the decisions of the higher Courts indicate that present prices should be used in making a valuation of land as well as of structural property, and that the present practice generally gives to the corporation as to the individual the so-called unearned increment due to appreciation in land values.

"This is incompatible with the fundamental principle that the corporation is entitled to a fair return upon the fair value of its property, because, if the land is valued at the increased price, and the appreciation in the value of the land is included in the accounting, the corporation would receive a sum in addition to a fair return, amounting to the appreciation in the value of the land."

On page 74, it reads:

"*Adaptation and Solidification* should be included in the valuation to the extent that they may have involved the expenditure of money. The increased value due to adaptation and solidification which has resulted from the action of the elements and natural causes without the expenditure of money should not be so included."

These quotations, and others which might be made, show the intention of the Committee deliberately to exclude from its valuation of a carrier's property certain distinct items and elements of value, so that, as we have heretofore seen, the result of a valuation reached under its recommendations would confessedly not be "the value" of the property. Not being and not pretending to be "the value" of the property, of what service would its valuation be? So far as a given railroad property is concerned, it has already been shown that given the true actual value of all its property, tariff rates for it could not be predicated thereon. This being true, what could any one possibly do with figures not representing "the value" of the property?

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The error of the Committee in making its report is fundamental. On page 6, it is said:

"It has been well settled by the highest Courts that the owner of such property [public service property] is entitled to a fair return upon a fair value of the property utilized in or reasonably necessary to the service."

Without at this time entering any dissent to this statement, attention only is called to the fact that throughout the report, the Committee has made the fundamental error of misconstruing and misunderstanding the basic principle of its report as thus stated. In other words, the Committee, as clearly appears, has proceeded on the assumption that "fair value" means something less than real value, and, so proceeding, has sought to work out a rule, by which, in valuing property, a figure may be arrived at, which, although less than "the value" of the property, will be its "fair value". The Committee, departing from substance, has wasted its energies on shadows; for "value", "fair value", and "reasonable value" mean the same thing, under the Constitution. If, however, by a play on shades of meaning, it is held that "fair value" is something less than "value", then only the figures representing "the value" are of any use when the Constitution is invoked. "Fair value" means no more and no less than "the value" fairly ascertained. The use of the word, "fair", may imply a limitation to the extent that "the value" required shall not be "fictitious value" nor "excessive value"; but it is likewise a limitation to the extent that "the value" required shall not be less than "actual value". The value required is not that which is fictitiously or excessively high or low, but is the real value fairly and reasonably ascertained.

In the Minnesota Rate Cases (230 U. S.), referred to by the Committee, the Court, quoting from Smyth *vs.* Ames (169 U. S., 546), said:

"The basis of calculation is the 'fair value of the property' used for the convenience of the public", but proceeded immediately to add: "Or, as it was put in San Diego Land & Town Co. *v.* National City [(174 U. S., 739)]:

"'What the company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used for the public.'"

Thus, the Court clearly implied that "fair value" and "reasonable value" were, for its purposes, synonymous terms. A few lines farther on in the opinion of Mr. Justice Hughes, from which these quotations are taken, he proceeds to quote with approval a further statement from Smyth *vs.* Ames, to wit:—"What the company is entitled to ask is a fair return upon the value of that which it employs for the public convenience". Thus, "fair value", "reasonable value", and "value" are used by the Court as interchangeable and synonymous terms. Later, in the opinion of Mr. Justice Hughes in the Minnesota Rate Cases, he

uses the following language: "It must be remembered that we are concerned with a charge of confiscation of property by the denial of a fair return for its use; and to determine the truth of the charge there is sought to be ascertained the present value of the property." Thus emphasizing again that it is actual value that must be ascertained. Mr. Churchill.

The law requiring the ascertainment of "original cost to date", which requirement the Committee ignores, proceeds to say:

"Fourth. In ascertaining the original cost to date of the property of such common carrier, the Commission, in addition to such other elements as it may deem necessary, shall investigate and report upon the history and organization of the present and of any previous corporation operating such property; upon any increases or decreases of stocks, bonds or other securities, in any reorganization; upon moneys received by any such corporation by reason of any issues of stock, bonds, or other securities; upon the syndicating, banking and other financial arrangements under which such issues were made and the expense thereof; and upon the net and gross earnings of such corporations; and shall also ascertain and report in such detail as may be determined by the Commission upon the expenditure of all moneys and the purposes for which the same were expended."

The Committee, in its report, page 65, says:

"It is the view of the Committee that an excess of deficiency of past earnings of an unregulated property should not be considered in making a valuation of the property, except as they may be useful in determining the development expenses. The Committee has been led to reach this conclusion, not because it would necessarily be inequitable to consider such past earnings, and especially recent earnings, but there seems to be no rational basis upon which they can be included. They are important as indicating whether rates are too high or too low.

"If one attempts to consider past earnings in making a valuation, many questions arise, Shall the consideration include the earnings for 2, 5, or 10 years, and if so, why not for a much longer period?" * * *

One has but to contrast this view of the Committee with the law to which it applies, to see how hopelessly apart the Committee and the law are. The law requires all "earnings" and all "expenditures" to be ascertained. The Committee say these things should not be considered. The law would have them considered in ascertaining "original cost to date", but the Committee, as we have seen, would not obtain that cost. The "excess or deficiency of past earnings of an unregulated property" will be considered by the Commission, because the figures will be ascertained and will be of a period, in many cases, when there were no regulating commissions. Of what value, therefore, is the recommendation of the Committee that the Commission must not do that which the law says the Commission must do?

On the subject of depreciation, the report deals at length and most interestingly. There is much said in this connection which is of value and will be helpful hereafter in solving this problem. There runs

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throughout all that is said, however, the same fundamental error which pervades the entire report, and that error is the purpose to obtain a figure for "value for rate-making", in lieu of one that will be "the value" of the property. So understood, the Committee's recommendations for ascertaining depreciation cannot be considered. The law requires that, as to a given existing piece of physical property, there be ascertained the cost to reproduce it new, and that there be then ascertained the extent or amount to which that existing item has deteriorated from new, which will be the depreciation, and which amount, being deducted from that representing the cost new, will give the present value of the existing item as it stands. This is a simpler statement of what depreciation is than the process for determining its amount may be, but it is a statement wholly unsatisfactory to the Committee.

On page 33, the Committee sets up what it calls an Equal-Annual-Payment Method, which, it acknowledges, it is not feasible to comply with exactly; and, on page 34, it is shown that it is equal only because the Committee has shrewdly chosen for illustration property of 20 years' life, and an interest rate of 5 per cent. It is shown on the same page that it is not equal in any other combination. It is acknowledged, on page 36, that "it is impracticable to devise any rule which will provide for strictly equal annual payments if the cost of repairs and operation is included".

Yet this is just what railroads have successfully done for years with fairly average annual expenditures covering repairs, replacements, renewals, and including the various forms of depreciation. Hence, why should the Committee endeavor to set up a scheme primarily calculated for local water-works, pipe lines, etc., and try to make it apply to railroads which have been purposely designed so as to allow of economical replacements of their elements in such a manner that the railroad property as a whole may always be kept practically at about 100% efficiency and value? It seems that the Committee in seeking methods of bookkeeping has laid aside the practical and correct methods which have been in use for years, and which it has failed even to attempt first to disprove.

The report, page 32, says:

"A depreciation allowance is, in effect, a payment from the ratepayer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

On page 37:

"Assuming the Committee's views to be correct, that a depreciation allowance is a return to the corporation of a part of its investment in existing items of property, the money so returned should be treated as a part of the capital and used for any purpose for which the corpora-

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tion is authorized to use capital, namely: for additions, replacements, betterments, or the extinguishment of outstanding obligations."

On page 38:

"Depreciation allowances should be such that the corporation shall have received, when its property is worn out or become obsolete, neither 50 nor 150% of its value, but as nearly 100% as the best methods of determining the depreciation allowances will permit."

On page 49:

"It will be observed that the writers in all these cases ignore the fact that a depreciation allowance earned from the rate-payers is a return of a part of the investment, and that after that part of the investment is returned, it is neither inequitable nor confiscatory to base future returns upon the depreciated value, which is the remaining investment in the property."

It seems quite clear from these quotations, and from others that might be made, that the purpose of the Committee in dealing with depreciation is to obtain what in its judgment will be a fair "depreciation allowance", and not the actual existing depreciation as affecting present value; and that the end to be reached is not the value of the property in its depreciated condition, but the "remaining investment" in the property, after the original investment has been depleted by whatever depreciation allowance has been "earned from the rate-payers". Interesting as all this may be, can any one see any connection between it and "the value" of the property at the time it is being used for the public?

On page 6, the report reads: "The subject is clearly in the development stage, and any conclusions reached at the present time must be considered as tentative," and, on same page, it again reads:

"It is recognized, however, that the whole subject is in a development stage, and while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

The question is here presented as to the value for any purpose of views which admittedly are but tentative, and which, in addition, are concededly in violation of the law as declared and expounded by our highest Courts.

It is to be said in conclusion that though the report as a whole clearly indicates that it was written with reference primarily to such public service corporations as water and gas plants, it nevertheless contains much that is of value in the valuation of the property of common carriers. It discloses much thought on and consideration of the valuation problem, and contains many suggestions that will

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be helpful in the solution of this problem. It is so fundamentally wrong, however, in the theory on which it is prepared, that as a whole it has no value for those dealing with the property of carriers. As it is written, a careful and exact following of its recommendations would not result in any one knowing "the value" of the property appraised. The report designedly and purposely excludes "value" as an end to be obtained, and substitutes therefor a sum, arrived at by arbitrary methods, to be used as "value for rate-making".

If it was desirable for such a sum to be found, the suggestions of the Committee as to methods for arriving at it would be entitled to due consideration, though, of course, it would have to be admitted that the fixing of this sum, whatever it should be, is more of a Governmental than an engineering problem. No purpose, however, could be served by a special value for rate-making as applied to carriers, because "the value" of their property when known will not solve the problem of rate-making, either for them or by them.

The Committee's report ignores (*a*) the requirement of the law that "value", without regard to the purpose of its use, be ascertained; (*b*) that the "value" when ascertained may be used for many other purposes than the regulation of rates; (*c*) that even in rate regulation it is only "the value" of the property that can be used; (*d*) that the protection of the Constitution extends to "the value" of the property; and (*e*) that when in support of a charge of confiscatory rates, a carrier presents the figures showing "the value" of its property, the Government cannot defend by showing a "value for rate-making" which is admittedly less than "the value" of the property.

For these reasons, and because the railroads of the country must be valued under the law, and not in accordance with any man's theories of what the law and the Constitution should be, the report should be referred back to the Committee to be harmonized with the law and the facts; and as this Committee was appointed "to formulate principles and methods for the valuation of railroad property and other public utilities", many portions of its present report should be revised, especially its Summary, Paragraphs No. 1, "Report Limited", etc.; No. 2, "General Principles"; No. 14, "Basis for Rate-Making"; No. 16, "Equal-Annual-Payment Method"; No. 19, "Appreciation"; and also the details of some other paragraphs. Further, these summary paragraphs should not be written as conclusions. On the other hand, as heretofore stated, the whole report should be changed so as to make it one of progress, from an engineering standpoint, and the Committee should be continued so that it may keep in touch with the reports of valuation, and with decisions of the Courts, as they are made from time to time, bringing to the attention of the members of the Society new features as they develop on this great subject, which, as the Committee states, is so clearly in the development stage.

M. H. BRINKLEY, ASSOC. M. AM. SOC. C. E. (by letter).—The Committee's report is a very comprehensive expression of valuation principles. The Equal-Annual-Payment Method of computing depreciation is both practical and logical. However, the decision of the Committee in regard to the use of depreciated value for rate-making may be supported by other considerations in addition to those given in the report. Mr. Brinkley.

In accordance with the agency theory of corporate control, the true relation existing between the public and the utility is that of principal and agent. The utility has a franchise which, in effect, is a contract between the two parties. In the very nature of the franchise, the utility has the right to do certain things which are ordinarily only vested in the State. The regulatory power of the State over rates follows naturally, because the utility is its agent.

The Courts have uniformly held that rates shall be reasonable, taking into consideration the value of the property. By "value" is not meant capitalized income, for if value is based on rates and rates on value, rates are always reasonable. Being denied the use of capitalized income, which is the real value of the unregulated property, the use of the actual investment or investment cost is the next step. The use of investment cost follows naturally from the agency theory.

If the utility is the agent of the public and the regulatory power is exercised from the beginning, the actual investment is the value on which rates can be predicated. However, the investment cost is not often obtainable, and, in lieu thereof, physical value is taken, as the investment was the physical value when the utility was new.

The physical value will have depreciated with the age of the utility. It must be assumed, however, that repayments have been made, either by unwarranted dividends or through earnings being placed in additions and betterments.

Practically all the utilities in the past have been unregulated. The adequacy of rates for the repayment of capital on account of depreciation has been under their control. If they have not been repaid, is it the fault of the public? The last paragraph of the Committee's quotation from the Knoxville water case is very apt in this connection.

If dividends by good management could not have been sufficient for repayment of lost value through depreciation, then the stockholders have simply made an unfortunate investment, as other utilities are not condemned for paying excessive dividends. Unearned dividends cannot be capitalized, and this applies to dividends which are insufficient to repay loss due to depreciation.

It has been stated that depreciation is a liability of the stockholders. Then, if true, replacements should be paid for out of dividends. Permanent depreciation has been called deferred maintenance, but the company will never again need to go to the expense of placing the property

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in 100% condition, because the life of all items is not the same. In past years there has been depreciation without any corresponding maintenance.

The simplest illustration of value is sales value, or value in the markets of the world. In the sale value of any object or unregulated property, the condition or depreciation is taken into account. The revenue being derived might also be considered, but the foundation value or physical value is the depreciated or present value. In the present value, the appreciation of some elements, as well as the depreciation of others, is accounted for. Justice is certainly done if they are both considered.

In basing rates on present value, the utility is not simply viewed as an aggregation of inert physical elements which have depreciated with age. On the other hand, it is seen as an operating property which is yearly taking out of earnings amounts sufficient to repay capital for depreciation. It is repaying this uniformly with the least drain on the earnings of the utility. The Straight-Line Method does not do this, as shown by the Committee, as the earnings have to be larger in the earlier than in the later years; but the Equal-Annual-Payment Method brings about the repayment in the most efficient manner, because it takes place on a compound-interest curve. The earnings on the remaining investment are the complements of the yearly repayments, and their total amounts will be the same each year if the interest rates are the same. As it happens, the actual depreciation takes place approximately in the same way. It is small in the early years of the life of the items and increases yearly to the end. It has been stated that, on account of the fact that railway companies are in 100% operating efficiency, they should be valued at cost of reproduction, which in this case means reproduction value as generally used. Does 100% operating efficiency mean that a utility structure is from 0 to 100% more valuable than one of the same age, life, and original cost which is being used for other than public service purposes? If there is any such additional value, it is a non-physical or intangible value, due to its use by a utility. In that case, however, the non-physical value would always amount to the actual depreciation. This value would then be greater in an old structure than in a new one, which is absurd.

In regard to operating efficiency, it must also be stated that we are not making a valuation of the efficient management of the owners, but are estimating the actual physical value of the property, and, in finding the physical value, the operating efficiency cannot be considered. Operating efficiency should only be considered in the rate of return allowed on the investment. It should there be given due consideration.

The advoctes of reproduction value for rate-making purposes contend that it is the nearest approach to original investment; but they fail to take into account that the property has been operating since

originally constructed, and, in finding the investment, the average earnings to date should be considered as the property has depreciated while producing earnings. This is on the basis of the agency theory exclusively. On this basis, if right of way for railways is included at present values, its increase in value from original cost should be counted as earnings. If this is not done, it should be placed at original cost. Mr. Brinkley.

If right-of-way multiples for railways are used, as advocated by the Committee, they are being taken on values which are in great part produced by the existence of the road itself. It would be very difficult to calculate what the values would be if the railway did not exist. The only alternative seems to be to take it no higher than the present market value of adjoining land, as this is the only measure of value which we have. However, if the actual cost was greater than the present market value, it should be given due consideration.

In the foregoing discussion, the views set forth are only meant to apply to a property in which no depreciation fund exists. For a property which has a depreciation fund, the writer can only express agreement with the views so ably set forth by C. E. Grunsky, M. Am. Soc. C. E., in his paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates."* In that paper he showed that, if a depreciation fund existed of an amount equal to the actual depreciation of the property, the reproduction value should be taken as the value for rate-making purposes. This is shown to be consistent with the use of present value, where no depreciation fund exists, as was shown by the writer in his discussion† of the paper by William J. Wilgus, M. Am. Soc. C. E., on "Physical Valuation of Railroads."

For a property with no depreciation fund, reproduction value could be used with the same result on earnings and rates as the use of present value, if the cost of replacements to the extent of interest on the depreciation is paid out of dividends. The net return to the stockholders is the same in the two cases, but the rate of return is different on the different values.

As stated by the Committee, development expense starts after the utility begins to operate. It cannot be considered in reproduction value new, because it is not incurred during construction, but during the early years of the plant's life, after it begins to depreciate. To a certain extent, it offsets depreciation in calculating present value. It could be included by allowing liberal value for interest and commissions.

When one contemplates the rate regulation problems, and the adjustment of rates with physical valuation, as the main basis, anything but an approximate solution for competitive utilities, such as railways,

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 244.

Mr. Brinkley.

seems hopeless. Under the circumstances, public ownership of railways and other utilities appears to be inevitable at some time in the future. If it comes, it will not be on account of the possibility of lower rates, but the desire of the public to have better control of public agencies, whatever the cost.

Mr. Stone.

H. M. STONE, M. AM. SOC. C. E. (by letter).—The title of the report as submitted is "Valuation for the Purpose of Rate-Making", while the Committee appears to have been appointed "to formulate principles and methods for the valuation of railroad property and other public utilities".

It should be distinctly borne in mind that methods of estimating depreciation on gas, water, and electric light plants, or similar public utilities, are not the same as those used for estimating on railroads. The railroad property is in the open, can be readily examined, and is not allowed to run until it is worn out, but is constantly in the process of renewal. In gas, water, and electric light plants, there may be no competition; they serve all the requirements of a community, quite in contrast to railroads which have to furnish equal service at the same price, and in nearly all cases are strongly competitive.

Railroad rates are not based on the value of the property used to render the service, and it is well understood that such rates are based, or substantially depend, on many things, among which may be mentioned: (*a*) value of service; (*b*) competition between carriers, localities, and commodities; and (*c*) cost of service. Carriers between competing points must maintain the same rates, and no shipper, locality, or class of traffic can be given any undue advantage or be subjected to any undue disadvantage. Rates, from different sources of production of the same or like commodities to the markets therefor or to the places where the same are consumed, must necessarily bear a certain relation to each other.

The report of the Special Committee ignores all the considerations on which railroad rates are, in fact, based, and erroneously assumes that they are, or can be, based on the value of railroad property.

The report is only a discussion of valuation "for the purpose of rate-making" (page 70). It asserts in substance that the depreciated value of the property should be the basis for rate-making, and proceeds on the theory that the depreciation allowance is in fact a repayment in installments by the rate-payer to the corporation of a part of the investment in particular items of property. It seems to be the opinion of the Committee that the sum said to be so repaid should not thereafter appear in the valuation of such items.

Applied to the valuation of a public service plant, like a waterworks, owned by a municipality, or enjoying an absolute monopoly, the report contains suggestions that may be useful in the work of

appraisal or in determining what rates, under the circumstances, ought to be fixed, but it is utterly inadequate as a guide for the work of ascertaining the value of railroad property, and the writer believes that certain statements therein are imperfect and misleading. Mr. Stone.

The Act of Congress directing the Commission to value railroad properties does not specify the purpose of the valuation, and any assumption that it is "valuation for the purpose of rate-making" is not justified.

It is a mistake to assume that either the cost of reproduction new or the cost of reproduction less depreciation may be used as a measure of, or sole guide to, the value of railroad property. The Valuation Act requires the Commission to investigate and report the value of all property owned or used by the carrier, and directs it to ascertain and report in detail, as to each piece of property, certain facts which are, or may be considered as, evidence bearing on value, among which are: (*a*), the original cost to date; (*b*), the cost of reproduction new; (*c*), the cost of reproduction less depreciation; (*d*), other values and elements of value; (*e*), the history and organization of the present and of any previous corporation operating the property; (*f*), any increases or decreases of stocks, bonds, or other securities; (*g*), the moneys received by reason of any issues of stocks, bonds, or other securities; (*h*), the syndicating, banking, and other financial arrangements under which issues were made, and the expense thereof; (*i*), on the net and gross earnings of such corporations; (*j*), the expenditure of all moneys and the purposes for which they were expended.

The writer is of the opinion that no one of these items of evidence is to be taken as the measure of the value of railroad property, but that it is expected that the Commission will take all into account and give to each its proper weight in the ascertainment of the value of the property.

The report (page 6) proceeds on the theory that "The valuation is for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more". Rates may be fair and reasonable and still be high enough to yield a substantial surplus after the payment of operating expenses, including maintenance and proper allowances for depreciation, and after the payment of a fair return on the full value of the property. As a general rule, railroad rates ought to be high enough to permit the accumulation of such a surplus in order to establish and maintain credit. In some instances, however, reasonable rates may not be sufficient to yield a fair, or any, return on the values of the property; but such instances are rare, and are usually due to peculiar or temporary conditions. The value of railroad property is not a means for the ascertainment of the reasonableness of rates, and, as a general rule, is not used as a guide by which rates are fixed. A railroad company is

Mr. Stone.

entitled to a fair return on the value of the property used in the public service, if that return can be earned by the application of rates which are not unreasonably high, and it cannot be compelled, without a violation of constitutional rights, to put in rates so unreasonably low that confiscation will result. The only cases in which the value of railroad property or the rate of return thereon is of importance as affecting rates are those in which it is claimed that Governmental authority has prescribed rates which are confiscatory.

It is always to be borne in mind that a rate may be sufficiently high to withstand attack on the ground that it is confiscatory, and yet be so low as to be unreasonable and contrary to a wise public policy.

On page 7 of the report it is stated in substance that

"Justice and equity require that in any regulation of rates fixing the amount which may be earned by a public service corporation there shall be taken into consideration two distinct features:

"First, the annual return covering interest and profit to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment.

"Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

It is respectfully submitted that this statement is inaccurate, and that the allowances are insufficient, because these allowances do not provide for any margin of safety or the creation of any surplus to serve as a basis of credit. It is in the interest of the public that the credit of the carriers be sufficient to enable them to procure funds necessary to provide facilities to meet the increasing demands for service.

Depreciation, as set forth in the report, is impractical, and almost wholly inapplicable to railroads. There are no tables of mortality that will give any reasonable results, nor would it be possible to make such tables for the bulk of the items of railroad property, nor are they a factor in the determination of rates.

The maintenance department of a railroad, at the close of a year, knows very closely whether or not there has been enough expended on the property to keep it up to its adopted standard. Any system of accounting by the use of mortality tables is relatively worthless as compared with actual knowledge as to the main facts in connection with the maintenance of a property, which knowledge is in the possession of those only who have had the maintenance in hand. The same holds true of equipment, that is, those having charge of the maintenance of equipment of a road know very definitely whether or not the equipment is kept up from year to year.

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The so-called depreciation is no more or less than the amount necessary for maintenance, and that amount can be known only by knowing the facts in the case for each property; it can never be ascertained by the use of mortality tables, which would introduce a mass of fallacious accuracy that would require continual adjustment.

Depreciation is not an amount paid back to the investor, which should no longer appear in the valuation, but it is the amount that it is necessary to spend to keep the investment intact, and, therefore, in any valuation made to indicate on what amount a railroad should be allowed to earn a return, it has no place whatever.

There are some instances where, as a matter of policy, a company should provide for some extraordinary renewal which it is apparent is approaching, by some special arrangement that will not throw the burden of this renewal into a single year; but that is a matter of expediency on the part of the railroad, and is not a matter of valuation.

On page 10 of the report it is in substance suggested that the cost of all obsolete items be deducted.

Without here expressing an opinion as to whether or not it is generally right to take into account obsolescence in the ascertainment of cost of reproduction less depreciation, the writer suggests that it is clear that the Valuation Act does not require obsolescence to be taken into account in the ascertainment of the cost of reproduction less depreciation. The depreciation there specified is to be ascertained by a comparison of the state of repair or physical condition of the item of property at the date as of which the valuation is made, compared with its condition when new.

On page 13 of the report it is suggested that, "An engineer in ascertaining a value for rate-making must take into account whether or not the property, although owned by the corporation, is in use."

It is not necessary that all railroad property must be in actual use on the date as of which the valuation is made, but there should be included all property acquired and held by the company, which it is expected will be used in the future. For example, in the development of a terminal in a large city, several years may be required in order to secure the needed land and to perfect arrangements with public authorities. Property acquired for immediate use and actually being prepared therefor at the date the valuation is made certainly ought to be included. It may be difficult to state a general rule to cover this point, but it is certainly unfair to make "actual present use" the test.

The attempt to segregate land into different classes, according to its use or proposed use, cannot properly enter into a just valuation. The owners of railroad property have decided that the acquirement of certain lands was a proper course to take. To treat land donated by the Government as of different value than other land, because of

Mr. Stone.

such donation, is unjust. As a rule, such land has been paid for several times over through the medium of special service rendered by the railroad to the Government. It cannot be said that some arbitrary strip of land has one value, and that another strip, possibly immediately adjoining, has a different value, basing such value on the apparent or present use.

On page 15 of the report it is suggested, "that valuations should be based upon the physical conditions existing at the time the various portions of the property were built, but at the prices prevailing at or near the time of the valuation."

The Special Committee is opposed to including in the estimated reproduction cost that of laying gas mains or water pipes in a street the expense of tearing up and replacing pavement which did not exist at the time the mains or pipes were laid, and seems to hold that such expense is no part of the present value of the mains or pipes, and apparently for that reason reaches the conclusion that the same should not be included in the estimated reproduction cost. It seems to the writer that this results from assuming that every figure which is used as "evidence of value" must be the same as that which "expresses the value". There is no doubt that in case of present reproduction the pavement would have to be torn up and replaced. Reproduction cost is not synonymous with value, but merely one fact or item of evidence to be used in ascertaining value. The engineering conception of reproduction should be followed throughout, and should not be modified to correspond with varying conceptions of value or to meet the supposed requirements of justice in particular instances. It is for the tribunal charged with the duty of finally ascertaining the value to give to the reproduction cost figure and to each of the other established facts such weight as they are entitled to in the light of all the evidence.

Again, on page 17 of the report, it is said:

"If after a pipe has been laid in a street new grades were adopted, the reproduction cost of the pipe should be estimated on the basis of the original grades, and if the regrading required a raising or lowering of the pipe already laid, the cost of such work, if not charged against the rate-payers as a part of the current expenses, should be included in the reproduction cost."

This statement confuses original cost with the conception of reproduction cost. If the property were presently to be reproduced, the cost thereof might not be affected by the fact that pipes had been raised or lowered since original construction, and it is not perceived how the source of funds expended for laying or raising or lowering the pipes would affect the present cost of reproduction.

On page 31 it is said:

"The Committee has already suggested that many principles and methods applicable to a normal property are not applicable to a 'losing

venture'. For instance, the 'losing venture' is not entitled to what would otherwise be a fair return on the fair value of the property. It has to be recognized in such a case that the investor must accept the resulting losses and such rates as the services rendered are worth, and to be recognized in such a case that the investor must accept the resulting earnings and not on other methods of valuation." Mr. Stone.

Undoubtedly, a railroad company cannot maintain rates that are unreasonably high merely because its enterprise is not a financial success, but it is difficult, even in the case of a "losing venture", to see how rates or revenue can be used logically to determine value, if that value is to be used as a basis for rates. The conception of the Committee seems to be that the rates are to be fixed at what the services are worth and that value may be based on earnings derived from such rates. This idea ought not be confined to "losing ventures". A railroad property so located, constructed, and operated that, at rates measuring the fair value of the service, it can earn an ample surplus, after the payment of operating expenses, together with a fair return on value, is certainly worth more than the cost of reproduction less depreciation, and such properties are usually worth more than the undiminished cost of reproduction new. It is unreasonable and unfair to limit the value of such a property to "The Depreciated Value" (page 72), and then to fix rates so low as to be sufficient only to pay return on such value.

The writer, of course, recognizes the fact that a railroad may be located and built so that it may be worth even less than cost of reproduction less depreciation, but such instances are unusual.

On page 32, it is said:

"A depreciation allowance is, in effect, a payment from the ratepayer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

It is always proper to include in operating expenses a sum sufficient to maintain each element of the property in proper repair and to replace its worn out parts. This "depreciation allowance" is not a payment to the corporation of a part of its investment in any proper sense, but is a just part of the operating expenses.

On page 41, it is said:

"There is a fundamental objection to depending upon the personal judgment of appraisers, as this method determines only the total depreciation of the property and not the amount of the depreciation allowance which the corporation is entitled to earn, unless it is to be assumed that the difference in the amount at which the property is appraised from time to time furnishes a measure of the amount of depreciation which should be allowed during the period between such appraisals. It is obvious that very erratic results would be obtained by such a method, especially if different persons acted as appraisers in different years."

Mr. Stone.

This statement seems to fail to make a distinction between the ascertainment of the value of the property in its present condition and the amount of money required to keep up the property and to renew its worn out parts. The value of the property is one thing and the amount of money necessarily included in operating expenses to repair it and to renew its parts is another and entirely different thing—one is the appraisal of the value and the other is the ascertainment of an element of operating expenses.

On page 50, under the caption, "Appreciation", the report seems to indicate that it is the view of the Committee that railroad companies are not entitled to the increase in value of their right-of-way and terminal lands; it is said that such lands "sometimes increase in value so rapidly that the question arises as to the justice of basing rates upon the present value of the land rather than its original cost."

The writer understands it to be settled by the highest authority that, if it can be earned on reasonable rates, a railroad company is entitled to a full and fair return on the value of the property at the time it is used in the public service. It is known to everybody at all familiar with railroad affairs that, notwithstanding enormous increases in value of terminal lands, rates have constantly decreased. It is not necessary to confiscate the increase in value in order to have reasonable rates.

It is stated in the report:

"The Courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value. The principle involved in the two cases is the same, and there is no reason to think that its application to both would not be supported by the higher Courts."

"The sum necessary to offset depreciation" is just as plainly an operating expense as the cost of labor or fuel required to operate the locomotives. The present value of the terminal lands of a terminal company is property, and the company is entitled to pay for its use in public service and is not limited to a return on that portion of the value which existed at the time of acquisition. Railroad property is private property, subject to regulation to prevent excessive charges and discrimination. As the writer understands the suggestions of the Committee, it is that the appreciation in the value of land be considered as an offset and payments of depreciation of other parts of the property. It is certain that the company cannot use the increase of terminal land value directly to maintain its roadbed or to repair or replace worn out equipment or structures. The appraisal of land at present increased value and the allowance of a return thereon to take care of depreciation (page 50) not allowed for in operating ex-

penses—as it should be—is unfair, and amounts to confiscation of such return or increased value. Mr. Stone.

On page 67, it is stated:

"Embankments and the slopes of cuttings may become more stable on account of the growth of grass and weeds, and may therefore be considered to be more valuable, but such value as this, which is not the result of expenditure by the corporation, should have no place in the valuation of the property."

This ignores the well-settled proposition that the railroad company is entitled to full present value of its property. The principle on which this suggestion rests would deny to the company the value of lands and other property granted or donated as an inducement for the building of the road; it would deny to the company the value of all property created out of surplus earnings; it would deny to the company the increase in value of wooden structures, due to the rise in price of lumber. In short, it would leave it within Governmental power to limit the company's earnings to a sum sufficient to pay a reasonable rate only on the money actually contributed directly by owners to create the property, and this without any guaranty of any return whatsoever on such sums.

S. WHINERY, M. AM. SOC. C. E.—This report is confined mainly to the question of valuations for rate-making purposes, and deals with a number of factors under the general head of depreciation. Mr. Whinery.

To the speaker's mind, it is a serious question whether the use of this word, depreciation, to cover a number of separate elements or factors, as is now the practice of engineers, and to some extent of the Courts, is necessary or wise, especially in so far as discussions or negotiations with public officials and the public at large are involved.

The careful observer, in these times of business and industrial agitation and confusion, cannot but note that no small part of the current misunderstanding, dissent, and distrust has its roots in the failure of different classes of society to understand each other. This is especially true as regards the ability of the business community and the public at large to grasp and appreciate the views and statements of technical men and the doctrinaires. This is often due not so much to the existence of really conflicting views, as to the fact that these views are not presented in language and terms that all understand alike. This applies to some extent to the technical forms of thought and expression used by the engineer.

In professional investigations and intercourse among themselves, it is entirely proper that engineers should resort to such technical terms as have come to be recognized as having an accepted meaning by them, but would it not be better and wiser if, in communicating with those outside the Profession, and in dealing with public and semi-public matters, they would confine their professional vocabulary,

Mr. Whinery.

as much as possible, to such words and to such meanings of those words, as are recognized and understood by the public?

Take, for instance, this word, depreciation. Its ordinary meaning is loss or decrease of value or of utility; but, as engineers, we have apparently read into it, possibly unintentionally if not unconciously, meanings that are, stricting speaking, quite foreign to it.

Thus, an annuity, the real purpose of which is to create a fund to repay capital at a stated time, is not equivalent to, and may not be a correct measure of, depreciation in the primary meaning of that word. In the case of public service contracts the annuity or sinking fund should logically mature at the time the contract expires, without regard to the life of the plant. Again, cost of maintenance is not depreciation but its antithesis—a provision for preventing depreciation.

The fact is that engineers themselves are somewhat at sea as to the definition and use of the word, and differ quite widely in their methods of dealing with the subject and in the several elements which they think should be included under that general head.

The Committee would render a valuable service to the Society and to the Profession, if it would give this matter careful consideration, recommend an acceptable definition of depreciation, and limit the cost-factors that may properly be included under that name.

In the equal-annual-payment plan proposed by the Committee under the general head of depreciation, the elements that make up the annual payment are not very clearly stated. Apparently, they include a sinking-fund payment to repay capital, interest charge on the capital invested, some part, at least, of the cost of repairs and maintenance, and possibly some provision for betterments. The result arrived at by this scheme may be wholly fair and reasonable for rate-making purposes, but the non-professional man will hardly be able, without more careful study than he is likely to devote to it, to understand how the annual payment is made up, and will be likely to look on it, or any other rigid depreciation formula dealing with several elements, with more or less mystification and distrust. There is evidence that not a few engineers who have given the subject no little thought, seem to be in a like frame of mind.

It may be argued with much reason that the whole subject of negotiating and establishing rates may be handled satisfactorily without any reference to depreciation, in the complicated sense in which it has come to be sometimes used.

A franchise for a public service is usually for a long term of years. Its purpose is to secure for the public a stated quantity and quality of service during its life. This requires the investment of a sufficient amount of capital to supply that service in a satisfactory manner. The party supplying the service is entitled to a fair and reasonable compensation therefor, and the object of rate-making is to determine

what that compensation should be. Now any intelligent public official, business man, or citizen knows, or may be readily shown, that a fair and reasonable rate must be made up of certain elements, the principal ones being: Interest on capital invested, operating expenses, repair and maintenance of plant, a liberal allowance for contingencies, a fair profit above ordinary interest, and, in the end, the return of the capital invested. Mr. Whinery.

There is here no mention of depreciation, nor is it necessary. The charge for maintenance takes its place. Obviously, the property must be maintained from the beginning to the end of the franchise in a condition to furnish properly the service required. If this be done satisfactorily, there can be no functional depreciation, or, at least, none from any proper point of view of the party receiving and paying for the service. The intrinsic or market value of the plant may fluctuate from time to time, as machinery is worn out and replaced, but these fluctuations do not affect the value of the plant for the purpose for which it was provided, nor call for any change in the rate of compensation, and they do not, therefore, interest the party receiving the service.

Notwithstanding, however, that the plant must be maintained in a condition to supply the service up to the very end of the contract time, its intrinsic or market value, at the end of the period, will be less than its original cost, and the difference between these two sums is depreciation in the sense in which the speaker would suggest that the word be used. Obviously, in the case of a perpetual franchise, there would never be depreciation in that sense, but the charge for maintenance would be sufficient to keep the property up to the necessary standard of adequacy.

Of course, the person, company, or engineer negotiating to supply the service may use such depreciation formula and other methods and devices as he thinks proper in computing the cost of maintenance, and must be prepared to translate his conclusions into facts and arguments comprehensible to the business man, but in all other respects the transaction will rest on plain business principles and practices which the average business man will readily understand.

The Committee seems to hold that the rate charged should include some provision for betterments and improvements. It will be difficult to convince the alert, clear-thinking, business man that such a provision is just. He will reason that if the equipment as originally designed and provided is adequate to supply the service contracted for, and is thereafter properly maintained, the cost of such maintenance being provided for in the rate, no additional capital investment will be needed to meet the full requirements of the contract; and that if additional or more expensive service becomes necessary or is wanted, supplemental contracts or franchises in which the question

Mr. Whinery.

of additional capitalization and, if necessary, a proper readjustment of rates can be considered, would be the proper and business-like way to deal with the matter; and that, as to improvements, they will not usually be made unless they promise to decrease the cost of production, and consequently increase the net profits to an extent that will make the improvements, in the opinion of the contractor, advantageous to him, under which conditions the party receiving and paying for the service should not be called on to bear their cost. In any event, it would be difficult to convince him that these items have any logical connection with depreciation.

What has been said assumes that we are starting with a new project and plant, the cost of which (*i. e.*, the capitalization) may be ascertained and demonstrated with reasonable accuracy. In the majority of cases, however, where the question of rate regulation comes up, an existing plant is involved, and the question of its value, as indicating the amount of capital invested, must be determined by an appraisal, and it may be said that in all such appraisals the factor of depreciation is so important as to be controlling; but, is this true? Does not the engineer or appraiser of wide knowledge and experience actually base his judgment on concrete facts and conditions rather than on deductions from any theory of depreciation?

In judging of the value of a machine or a system of machines, the real question is not what is its age or what work has it already done, but what work is it capable of doing in the future? We want to determine its expectancy of useful life from this time on, rather than its fair average expectancy of life when new, in order that we may subtract its present age from its estimated total life and assume the remainder as its future potential useful life. Its history may be useful to us in arriving at conclusions, but such evidence is of secondary value. It is useful to know that the average life of a certain kind of machine is, say, 20 years, but we are likely to make serious errors if we use that figure as a measure of the life of all such machines. That life should be measured in units of work and endurance rather than in units of time. This is all the more true because the element of proper maintenance is very important. We all know that, of two machines, duplicates in all respects, difference in care and maintenance under the same work may make the useful life of the one 100% longer than that of the other. How then can we be justified in applying any rigid rules of depreciation based on assumed or average life to individual machines? Must not the value of each be judged on the present evidence of its probable efficiency and durability, under the conditions of service to which it is likely to be subjected? We must remember, too, that unforeseen conditions of inadequacy or obsolescence may appear at any time, in defiance of all formulas,

and the weight to be given to their probability must be purely a matter of good judgment. Mr. Whinery.

The speaker's argument is not, however, that the appraiser may not consider the bearing of any theory or formula of depreciation, or any other data or evidence available, in arriving at the value of any machine or plant, though he believes the more experienced and capable a man becomes in appraisal work the less weight will he attach to theoretical or hypothetical considerations and the more will he rely on tangible practical evidence. The speaker, however, is convinced that reports of appraisals that are to go before non-professional men, such as public officials and business men, are not strengthened by refences to or discussions of complicated theories or rules of depreciation and conclusions that seem to be based thereon.

On the whole, it seems to the speaker that engineers have built up around this word, depreciation, in so far at least as it is concerned in rate-making, a complicated and unnecessary body of theory and speculation, not very useful to themselves, but well calculated to be a stumbling block in their dealings with the public.

PHILIP W. HENRY, M. AM. SOC. C. E.—This report is an interesting and valuable document, and the Committee is to be congratulated on the thorough manner in which it has dealt with this important subject, covering as it does so many of the elements which enter into this problem. The discussion on depreciation, however, is not clear, and the speaker is at a loss to know whether the Committee intends to express the opinion that, after a depreciation reserve is set aside, the investor is entitled to a full return only on the original investment less depreciation reserve, or whether he is entitled to a full return on the original investment. As various statements on this point are apparently not in accord with one another, it would seem that the Committee should express itself with greater exactness, for it must be remembered that any report adopted by this Committee will carry great weight, not only among engineers who are more or less familiar with the principles of depreciation, but also among public service commissioners and others who may not have made a study of the subject and who, therefore, may be misled by the apparent discrepancies in the report. That the members of the Committee had clearly in their own minds conditions under which the investor is entitled to a return on his original investment, and other conditions under which he is entitled to a return only on the "remaining value of property", is not doubted by the speaker, but these different conditions should be presented with such simplicity, conciseness, and clearness as to leave no doubt in the mind of the reader, however unfamiliar he may have been heretofore with the elementary principles which govern depreciation. Mr. Henry.

Mr. Henry.

As stated by the Committee: "There is no subject connected with valuation about which there are more diverse views than those relating to depreciation." In addition to this difference of opinion which exists in regard to depreciation itself, there is also a difference of opinion as to whether the same method of depreciation should be used for private corporations, where investors alone are interested; for public service corporations, where rates are regulated by public service commissions; and for railroads, where an endeavor is being made to fix rates in accordance with physical valuation. In the speaker's opinion, the same method should be used in all cases, for depreciation is one of the elements necessary to present a full and clear statement of the condition of the property; and whether this full and clear statement is for the use of the investor or of the public service commission is immaterial. In order that the investor may know the true value of his shares of stock, he must know exactly the condition that his company is in, and unless depreciation is properly taken into account, he cannot have this full knowledge. The same is true of a public service corporation, for, in order to fix rates, it is essential to know the true earnings of the corporation, and such earnings cannot be known unless depreciation is properly cared for. Therefore, whether depreciation is set aside by a mining company, the operations of which will inevitably come to an end (say, in 20 years); or whether it is set aside by a public service corporation or a railroad (the operations of which will continue indefinitely), the principle is the same in each case, and the same method of depreciation should be used.

It will be admitted that an investor is entitled, not only to a reasonable annual return on his investment (when properly and intelligently made), but also to receive back his principal at any time, whether he receives it through the sale of his property to other parties, or whether, as in the case of an exhausted mine he receives it from a depreciation reserve which has been accumulated out of earnings, and which when the mine is exhausted (assuming no scrap value), just equals the amount of his original investment. Such depreciation reserve, when properly set aside, guarantees the integrity of the original investment, whether the money is invested in a mine or in a public service corporation. If in a mine, the investor alone is interested in knowing that his original investment is maintained at a constant value, and that the return he receives in dividends, whether 5, 10, or 50% annually, is really earned. If the money is invested in a public service corporation, not only is the investor interested, but also the general public, which, by the rates it pays for the services rendered by the corporation, is interested in knowing that the property is being maintained out of earnings at a constant value (thus assuring equally efficient service as the plant grows older) and that, in addition, there are earnings sufficient to provide a reasonable annual return on the

money originally invested. If the earnings are greater than are required for these two purposes, then the public may be justified in demanding a reduction of rates. If they are less than sufficient to maintain the property at a constant value and to pay a reasonable return on the money invested, it will be impossible for the corporation to obtain the new capital necessary to maintain the efficiency of the plant, and to make such additions as are necessary to take care of the increasing business. If, therefore, a public service commission or other authorized public body establishes a maximum annual return for a public service corporation, it is important that both the commission and the corporation should know that, after paying operating expenses, including the cost of ordinary repairs and small tools, there remain earnings sufficient to pay the authorized annual return on the original investment and set aside in the depreciation reserve each year a sum adequate to maintain the property at its original value. In other words, the amount standing at any time in the depreciation reserve, plus the value of the property at that time, should equal the original value of the property (original investment), neglecting the influence which appreciation and other elements may have on the problem so fully presented by the Committee. It must be borne in mind that in this discussion the speaker confines himself solely to depreciation, without reference to other features of the Committee's report. If it is possible to arrive at general principles governing depreciation, applicable to all classes of corporations, the application of such principles to a given corporation will be comparatively simple, due allowance then being made for the effect which other elements may have on the problem. Mr. Henry.

Strictly speaking, the only true way to set aside a sum each year for a depreciation reserve is to make each year a careful valuation of that portion of the property subject to depreciation, and the difference between the value thus established and that similarly established the previous year, is the amount which should be placed that year in the depreciation reserve. Manifestly, such yearly valuation is impracticable, and it has been generally agreed that the simplest method for determining the depreciation is to assume a life-period for each part of the plant subject to wear or obsolescence, and to set aside each year a certain sum based on such life-period. Whether the investor in the assumed public service corporation is entitled to the authorized annual return on the original value or on the remaining value depends entirely on the method used in setting aside the depreciation reserve. For purposes of simplifying the discussion, it will be assumed that the depreciation reserve is invested in outside securities, although the argument holds good in case it is invested again in the property, in betterments and additions or otherwise, earning the same rate of interest as if invested outside.

Mr. Henry.

If the depreciation reserve is maintained by the Sinking-Fund Method, by which the sum set aside each year, plus interest compounded annually, amounts at the end of the life-period to the original value, the stockholder is clearly entitled each year to his full return on the original investment. On the other hand, if the depreciation reserve is set aside by the Equal-Annual-Payment Method, or by the Straight-Line Method, by which certain sums are set aside each year, totaling (without interest) the original value at the end of the life-period, the stockholder is clearly not entitled to a return on the original value, as he either receives directly the interest on securities in which the depreciation reserve is invested, or else the property receives it as an amount available for dividends.

To make this clear Table 18 has been prepared, assuming that the property has a 20-year life-period, valued when new at $100; authorized annual return, for example, of 7%; annual interest of 7%, for example, on funds standing in the depreciation reserve, such percentage being compounded annually to the depreciation reserve in the Sinking-Fund Method; but, in the other two methods, the interest being paid out each year either directly to the investor or to the property available for dividends.

TABLE 18.

	Sinking-Fund Method.			Equal-Annual-Payment Method.		Straight-Line Method.	
End of year.	Amount set aside each year for depreciation reserve.	Interest at 7% compounded annually.	Total yearly amount to depreciation reserve.	Total yearly amount to depreciation reserve.	Interest at 7% paid out each year to investor.	Total yearly amount to depreciation reserve.	Interest at 7% paid out each year to investor.
1	$2.44	$0.00	$2.44	$2.44	$0.00	$5.00	$0.00
2	2.44	0.17	2.61	2.61	0.17	5.00	0.35
3	2.44	0.35	2.79	2.79	0.35	5.00	0.70
4	2.44	0.55	2.99	2.99	0.55	5.00	1.05
5	2.44	0.76	3.20	3.20	0.76	5.00	1.40
6	2.44	0.98	3.42	3.42	0.98	5.00	1.75
7	2.44	1.22	3.66	3.66	1.22	5.00	2.10
8	2.44	1.48	3.92	3.92	1.48	5.00	2.45
9	2.44	1.75	4.19	4.19	1.75	5.00	2.80
10	2.44	2.04	4.48	4.48	2.04	5.00	3.15
11	2.44	2.36	4.80	4.80	2.36	5.00	3.50
12	2.44	2.69	5.13	5.13	2.69	5.00	3.85
13	2.44	3.05	5.49	5.49	3.05	5.00	4.20
14	2.44	3.44	5.88	5.88	3.44	5.00	4.55
15	2.44	3.85	6.29	6.29	3.85	5.00	4.90
16	3.44	4.29	6.73	6.73	4.29	5.00	5.25
17	2.44	4.76	7.20	7.20	4.76	5.00	5.60
18	2.44	5.27	7.71	7.71	5.27	5.00	5.95
19	2.44	5.81	8.25	8.25	5.81	5.00	6.30
20	2.44	6.38	8.82	8.82	6.38	6.00	6.65
Total.	$48.80	$51.20	$100.00	$100.00	$51.20	$100.00	$66.50

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From Table 18, it is evident that, under the Sinking-Fund Method, the investor is entitled each year to a return of 7% on the original value of $100. Under the other two methods, he is entitled to less than 7% on the original value, which apparently is what the Committee means by "the corporation must accept a return upon the depreciated value of such property". Referring again to Table 18, in the twelfth year, for example, under the Equal-Annual-Payment Method, the investor received $2.69 from the securities in which the depreciation reserve is invested (whether inside or outside of the property), and is, therefore, entitled to receive only $4.31 ($7.00 less $2.69) from the property itself, which is 4.31% on the original value of $100. In the twelfth year, under the Straight-Line Method, the investor receives $3.85 from the securities in which the depreciation reserve is invested, and is, therefore, entitled to receive only $3.15 ($7.00 less $3.85) from the property itself, which is 3.85% on the original value of $100.

From this comparison of methods it seems to the speaker that the Sinking-Fund Method is the simplest and best, but as (to use the words of the Committee) "the use of the words Sinking Fund appears to have misled some Courts and commissions", it may be better perhaps to call this method, as above exemplified, the Straight-Line-Interest Method, which describes the method of putting aside in the depreciation reserve an equal amount each year (2.44% as in Table 18) out of earnings, and adding to it the interest on the total amount in the depreciation reserve, whether it is invested in outside securities or in the property as betterments and additions, or otherwise. As it is difficult to find safe outside investments returning more than 4%, it is preferable, generally speaking, to invest the depreciation reserve in the property itself, where it will earn the same annual return as the original investment.

From the foregoing, as well as from the report of the Committee, it is evident that depreciation is not susceptible of exact mathematical treatment, inasmuch as it is based on the life-period of the various parts making up the property, or on the condition of each part in any one year. As both the life-period and the condition of any property or of its component parts are purely estimates, on which no two engineers will agree in detail, it is evident that in setting aside a depreciation reserve, refinement is unnecessary, and the method which will best accord with present methods of accounting and which can be most easily applied is the one which should be adopted. No matter what method is adopted, and no matter how conscientiously and intelligently it is applied, it is quite likely that an experience of 5 or 10 years will show that the depreciation reserve is too great or too small, and, therefore, that a change should be made in the percentage set aside each year. For this reason it may be advisable at convenient periods to make a valuation of those parts of the property susceptible to depreciation, in

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order to know whether or not the amount in the depreciation reserve equals the original value less the present value, as it should do—leaving out of consideration appreciation and all other elements which may affect the problem.

Depreciation reserve, as an account on the balance sheet, may stand alone as a liability on the credit side, or it may be shown on the debit side as a deduction from the asset, plant account. The speaker prefers the former method. No matter how it is shown on the books, or whether or not it is invested in outside securities, it should be reserved strictly for the purpose for which it has been established, and should never be considered as applicable to dividends. If it is found to be larger than conditions warrant, the amount set aside each year may be diminished or omitted altogether for a few years; if too small, the amount set aside each year may be increased.

As a matter of bookkeeping, at the end of each year profit and loss will be charged, and depreciation reserve credited, by the amount to be set aside, on each item of plant account, figuring the same percentage each year on each item, depending on the life-period and the rate of interest borne by the funds in the depreciation reserve. The depreciation reserve will also be credited at the end of each year with the interest earned during the year by the securities in which it has been invested. If it is merely a book account, and the funds have been used in the business for betterments and additions or otherwise, the depreciation reserve will be credited by interest on its average balance for the year, figured at the same rate which is being paid on the original investment, or 7% in the case which has been assumed.

On the debit side of the balance sheet, as an asset, will be found an amount exactly balancing, as an aggregate, the depreciation reserve as a liability on the credit side. If the depreciation reserve is invested in outside securities, the account on the debit side may be called "securities, reserve fund for depreciation". If the depreciation reserve is invested again in the property, it will be found, at the end of the year, that on the debit side certain assets, such as betterments and additions, accounts receivable, cash, etc., will be greater, as a total, by the exact total amount added that year to the depreciation reserve. During the year, from time to time, the depreciation reserve may be charged and cash credited (if a cash transaction) for any replacements, such as the purchase of a new engine to take the place of one which has been worn out or become obsolete; or these replacements during the year may be carried in a replacement account, and at the end of each year, the total of such account will be charged off to the depreciation reserve.

Under the Straight-Line-Interest Method (practically the Sinking-Fund Method), applied as already outlined, no question arises as to whether or not the investor is entitled to a full return on the original value of the property or only on the remaining value, as it does under

the Equal-Annual-Payment and Straight-Line Methods—thus making the latter two methods more difficult to apply. Another advantage of the Straight-Line-Interest Method is that the accounting is much simplified, and the only problem—not an easy one, however—is to establish the life-period of each item entering into plant account. Mr. Henry.

CHARLES HANSEL, M. AM. SOC. C. E.*—The Special Committee of the Society appointed under the resolution set forth on page 102 of the February, 1911, *Proceedings,* was instructed as follows: Mr. Hansel.

"*Resolved,* That a Special Committee of seven be appointed by the Board of Direction to formulate principles and methods for the valuation of railroad property and other public utilities, and to report to the Society at the next Annual Convention."

This resolution was adopted unanimously at the Annual Meeting of the Society on January 18th, 1911. It was offered by the speaker after he had stated that, in all the papers submitted on the subject of valuation of public utilities, there had been no definite outline of the proper principles and methods to be pursued in the doing of the engineering work necessary to the enumeration and valuation of the various physical elements, and that the papers and discussions were confined principally to questions of law and economics.

In offering the resolution, the speaker felt that it was clearly understood by the members present that, as an engineering body, the Society should take up the question of valuation as it relates to the gathering and compilation of engineering data. The title of the Committee's report, "Valuation for the Purposes of Rate-Making", does not appear in the resolution, either in precise language or in terms. The report deals mainly with legal and economic questions, devoting some 47 pages to the subject of depreciation; 13½ pages to land; 2½ pages to appreciation; and about ½ page to adaptation. There is no part of the report which deals with the problems attendant upon the gathering of field data. It touches upon preliminary expenses, engineering, general contingencies, and the like; all of which are provided for in the "Classification of Road and Equipment Account" of the Interstate Commerce Commission. It seems, therefore, unnecessary to discuss the propriety of including these items, as they have already been recognized and admitted by the highest accounting tribunal in the land.

It is always desirable that the engineer broaden his knowledge through the study of legal and economic questions. However, it appears to the speaker that the first chapter of the Committee's report should be devoted to the engineering questions involved rather than to a theoretical discussion of depreciation, amortization, and other accounting theories, which are important considerations of future accounting, and not aids to the determination of the present value of the physical elements or the whole property value of a corporation which

* This discussion was presented for Mr. Hansel by Carl Tombo, Assoc. M. Am. Soc. C. E.

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has been in operation for some years. The speaker holds that there is no mathematical rule for determining depreciation, although it is quite proper that some empirical rule be adopted in accounting methods, so that the accrued replacement needs of the future may be anticipated.

In the determination of the present reasonable value of the segregated parts of a railroad or other public utility, we are not assisted by any formula of depreciation to be applied as an accounting method; and it is not likely that, in the valuation of any important railroad or other public utility, the depreciation for any purpose will be calculated by rule, but rather by investigation and observation.

The Chairman of the Committee has advised the speaker that, owing to the intricacies of the whole subject, "it was decided to take up first only one branch of the subject of valuation". It would be inferred from this statement that this report of the Committee is but one of several which it expects to make.

The Committee, no doubt, is fully advised as to the requirements of the "Federal Valuation Act", which became a law after the Committee was appointed. This Act specifically sets forth the principles and methods which shall govern the valuation of the railroads of the United States. Therefore, there is no longer need of speculation as to such principles and methods, so far as railroads are concerned. This "Federal Valuation Act" does not even suggest that the results to be obtained under its specifications are to be used for rate-making. The "Federal Valuation Act" calls for (1) "The Original Cost to Date"; (2) "The Cost of Reproduction New"; (3) "The Cost of Reproduction Less Depreciation"; and (4) "Other Values and Elements of Value". Consequently, any theory of valuation to be applied to railroad property must consider, not only the physical elements, but also such "other values and elements of value" as affect either the physical elements or the total value. Neither the Federal Government nor any of the several States has attempted to make valuations of railroad property for the purposes of rate-making. The carriers, in some instances, have found it necessary to produce evidence of the investment in the physical elements of their respective properties, in order to defend them from the enforcement of rates which they deemed would be confiscatory.

Public utilities, other than railroads, generally sell only one commodity or service, the price of which—especially in the case of gas, electric, and water utilities—is controlled by ordinance or the sovereign power of the State. Such public utilities are local, exist by the grace of special franchise, and are generally monopolistic. As contrasted with these public utilities, it is well known that railroads sell a great variety of service in competition, and are under regulation by the Interstate Commerce Commission. In the case of a railroad, there is a very great variety of rates, involving, in whole or in part, identical

stretches of the property. In the speaker's opinion, therefore, it is unreasonable to suggest that there can be a determination of each of the thousands of different rates which obtain on a carrier's property by attempting to fix the value of the physical elements of the whole property. Mr. Hansel.

It is submitted that the report is based on a fundamental error, *viz.*, that railroad rates are based on the valuation of the property of the railroad company. It will be seen, from an examination of decisions of the Interstate Commerce Commission, of the Courts, and from leading textbooks on the subject, that the rates of a railroad company are based on:

(*a*) Value of service;
(*b*) Commercial and competitive conditions;
(*c*) And in some slight degree, though very remotely, on the cost of service.

The first railroads in the United States were merely temporary paths to greatness. Modern methods of construction were not applied in the original building of any of these earlier railroads. They were built along the line of least resistance, without regard to grade or curvature. The policy of the original constructors was to build as cheaply as possible. To cheapness of construction was sacrificed shortness of line and flatness of grade and curvature. This policy, perhaps, was the better one at the time it was adopted, because the country was sparsely settled, and the most sanguine failed to grasp the growth that was to follow.

The problem with the railroads to-day is not so much how to get more business as how to carry the existing traffic at low cost. The economic demands of modern traffic require, on the part of such railroads as the Pennsylvania, the Baltimore and Ohio, the Vanderbilt Lines, the Harriman Lines, and others, that they expend enormous sums of money on improvements which are, in a large measure, merely corrective of original defects in location and construction resulting from former methods and from enforced economy due to the high price of money and the uncertainty of earnings.

The cost of operation per train-mile is approximately the same whether the train has a capacity of 200 or of 400 tons. It is evident, therefore, that—given the same rate and tonnage—the earning capacity of a railroad which, by reason of its flat grade and easy curvature, can haul 400 tons per train on a given tonnage would be much greater than the earnings of another railroad which, with the same equipment, could haul only 200 tons; and it is usually found that the railroad capable of carrying the greater train load, with a given power, costs much less to build than one of smaller train capacity. As the contractional relations between railroads influence the tonnage very mate-

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rially, it is quite possible that the cheaper railroad, having the greater train-load capacity, would be short of business on account of not being as favorably situated to receive tonnage from other roads.

There are cases where it would be economical, from a broad public policy, for a railroad to abandon its line entirely and build in different territory between two given cities, thereby saving in operating cost for carrying the same tonnage between the same termini. In many cases this saving of cost of operation capitalized would more than equal the cost of building a new line. This change cannot always be accomplished, for the reason that towns and industries must be served. Consequently, the high cost of carriage between the termini must continue; and, if a new organization constructed a line in the easier territory suggested, and secured satisfactory terminals and traffic arrangements, it is apparent that it could conduct its business at a smaller cost and with greater expedition. The securing of business, however, depends not only on convenient terminals and efficient organizations, but also on the contractional relations with other railroads which are in a position to give or withhold business; and, though the value of the physical elements of the least costly railroad, as represented by the real estate and the tangible personal property, may be much less than that of its competitors, the commercial value may be much greater by reason of its maximum train-load capacity and its minimum operating expense.

The value of low gradients and the following conditions actually exist in one of the Southern States:

"A" represents the train load of a Cooper E-50 on a grade of 15.84 ft. to the mile, or 0.3 of 1%; load, 3 022 tons.

"B" represents the train load on a grade of 53 ft. to the mile, or 1%; load, 1 322 tons.

"C" represents the train load on a grade of 60 ft. to the mile; 1.136%; load, 1 191 tons.

It is understood that the speed of the train is 18 miles per hour, the tractive force is 0.2 of the weight on the drivers, and the train resistance on the level is 6.5 lb. per ton.

It is evident that, if these three railroads have an equal division of business, and like rates, between two important cities which they connect, the net earnings of "A" will greatly exceed those of either "B" or "C". However, it is not likely that the division of business is equal, and before a final determination of value is made, the actual tonnage hauled should be ascertained; and this current tonnage is frequently controlled through contractional relations with other roads, which may vary from time to time.

If the property investment or physical value of the parts constituting the three railroad properties cited were used to determine the

justice of rates, line "A" would be given a rate about one-half of that of line "C". Mr. Hansel.

Referring to page 6 of the report: The statement that "valuation is for the purpose of determining rates which shall be a limitation of charges to those which shall give a fair return" is entirely erroneous. It has been held by the Interstate Commerce Commission and by the Courts that rates which gave more than a fair return, might be entirely reasonable; and so, also, it has been held that a given rate which affords less than a reasonable return might still be a reasonably maximum rate. The only case in which the question of return as affecting rates is of the slightest importance is where a Governmental body proceeds to establish an entire schedule of rates, and where the carrier contends that the schedule as established affords less than a reasonable return on the value of its property devoted to the public use; in other words, asserts confiscation, and accordingly a violation of its constitutional rights. If the principle be asserted that is apparently contended for in this portion of the report, that rates must be limited to such as will secure a bare return on the value of the property, it is obvious that no private capital will go into a business where the profits are limited, and where there is no guaranty against loss.

In further argument that the title is a misnomer, and that there can be no valuation for the purpose of rate-making, attention is directed to the address* of Mr. Charles A. Prouty, Interstate Commerce Commissioner, delivered at the Annual Meeting in the Chamber of Commerce of the United States, at Washington, D. C., on February 11th, 1914. Mr. Prouty is now the Director of the work involved in the valuation of railroads under the "Federal Valuation Act", consequently, his words on this subject should be carefully considered.

The principles which govern the determination of the public-service value of railroads are, in many respects, quite different from those which apply to the public-service value of other public utilities.

It will probably not be denied that railroads are the most important factor of our internal material progress. They have been constructed in advance of all other commercial developments; and, indeed, in many cases, the constructors, the bankers, and the engineers were the pioneers —the explorers of the land, at present busy with industry which now requires for its refined conveniences those other public utilities such as water, gas, and electric plants, urban and interurban transportation, and the like. Thus, the railroads stand as a necessity to the material progress of the whole country, whereas the other public utilities are local, and are created out of the demands of a refined community dependent for its being on the facilities furnished by the railroads.

The railroads differ from other public utilities in construction and maintenance. The track of a railroad is made up of small units, which

*_Railway Age Gazette_, February 13th, 1914.

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can be currently renewed; and thus the track may be, and is, generally, kept at 100% efficiency, sufficiency, and adequacy. There is, in track and roadbed, no decrepitude or obsolescence beyond the reach of maintenance.

Stated in another way, the roadbed of the carrier, which is generally a large item of cost, is constantly appreciating through consolidation, enrichment, and growth of vegetation on slopes, thus protecting it from erosion and thereby reducing the maintenance. The track itself is constantly being renewed.

A pipe line is constantly depreciating; it is beyond the reach of maintenance; when it fails, through decrepitude or insufficiency, the investment in constructing the trench, laying the pipe line, etc., is lost; and there is practically little scrap value.

Each item of track has a different life, and the history of each is represented by a series of independent, recurring cycles, which together make up the whole and produce an approximately uniform average, which, measured by efficiency, is 100 per cent.

The "Valuation Act" requires us to apply depreciation to cost to reproduce. Therefore, we must determine a method of fixing depreciation as contemplated by the Act, though we are not apprised of the purpose to which such depreciated condition is to be applied. In making this deduction for depreciation of individual items, we do not acknowledge that by so doing we depreciate the property as an instrument of commerce, which, when maintained as in the case of the majority of the carriers, furnishes a constantly adequate and sufficient system for public service.

The Special Committee was appointed under a resolution offered in January, 1911. Since then the valuation of railroads and other public utilities has been discussed by engineers, economists, and various technical societies, also before the House and Senate Committees of Congress; and, on March 1st, 1913, there was enacted a law: that an act entitled "An Act to Regulate Commerce", approved February 4th, 1887, as amended, be further amended by adding a new section, to be known as Section 19a.

Notwithstanding the enactment of the Federal Valuation Law, no mention has been made of it in the report, although it says, on page 6:

"It is obvious that a valuation before a Court or public service commission must, in order to be sustained, be made in conformity with existing laws and the decisions of the higher Courts".

The speaker desires to direct attention to other criticisms of the report, which may be stated briefly as follows:

(1) Page 7: The third paragraph, referring to the allowance sufficient to provide for depreciation, is insufficient, in that it does not provide for an adequate surplus to serve as a basis of credit.

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(2) Page 11: The second paragraph on this page, which apparently eliminates the amount of investment as a factor in valuation, is most unjust, as it excludes entirely from consideration cost to date, and seems to assume that all investment which has become obsolete should be written off and no return allowed thereon.

(3) Page 11: The third paragraph, which recites that rates should be limited by a valuation, is absolutely impracticable in the case of a railroad company, for the reasons heretofore set forth.

(4) Page 32: There does not seem to be any justification in equity or in law for the theory apparently set out here, that the present value must exclude that part of the investment which has been replaced by amounts charged to operating expenses.

(5) Page 50: Criticism 4 applies also to the statement in the sixth paragraph of this page, with respect to appreciation and depreciation balancing each other. The statement also is apparently in conflict with the decision of the Court in the Minnesota Rate Cost, and the Consolidated Gas Case (212 U. S.); and apparently in conflict with the last paragraph on page 60 of the report.

(6) Page 67: The statement in the second paragraph, that value resulting from increased stability of embankments and slopes cannot be taken into consideration, would seem to be legally and equitably unjustifiable.

(7) Page 70: The statement in the fourth paragraph of this page is defective, in that it does not include any provision for a surplus as a basis of credit or for unforeseen expenses due to unusual destruction by the elements.

(8) Page 73: The recommendation on this page, as to treating appreciation, would seem to be neither good law nor equity. The report apparently concedes that it is not good law.

The Interstate Commerce Commission, together with its Engineering Board, Legal Advisers, and Accountants, has been engaged since June, 1913, in formulating specifications, rules, and methods of procedure for the guidance of all concerned in the valuation of the railroad properties of the United States. Up to the present time, its efforts have been directed toward the solution of engineering problems, thus illustrating the importance and difficulty of gathering engineering data. Although Commissioner Prouty, in his address before mentioned, sets forth clearly the various problems associated with the application of depreciation and "other values and elements of value", the Commission has refrained from dealing with the complex economic questions to which the Special Committee has devoted its report.

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Any attitude, of the Federal or State Governments, toward railroads or other public utilities, which tends to throttle progress is admittedly against sound public policy. As has been stated, it is necessary for the carriers of to-day to expend large sums of money in betterments, in order to reduce the cost of transportation in the face of advancing costs for labor, materials, and supplies.

Large expenditures have been made in the reduction of gradients for the purpose of increasing the trainload and reducing the operating cost. If the beneficial results of such expenditures are to be denied the carriers, does not such action throttle progress?

The reduction of gradients frequently involves a change in gradients on the same alignment and the moving of material at maximum cost. After the accomplishment of such work, the reproduction cost of that portion of the property affected by the betterment does not represent the cost under the actual conditions of construction; and, even though it did accurately represent the cost, it seems unwise to take from the carrier the results of his efforts to increase the safe and economical operation of the property.

It must be apparent, therefore, that any attempt to fix rates of carriage by measuring the investment in the physical property is against sound public policy, even if it were possible to carry out any such plan.

Is it then good public policy to say to the carriers that it is useless for them to attempt improvements which will provide safer and more efficient transportation? Will the carriers be able to finance additions and betterments, which frequently require the abandonment of property, if they are to be denied a reasonably attractive return on the new capital as well as on the property sacrificed to the cost of progress?

It is a matter of common knowledge, that recently a carrier, which for years has enjoyed the most efficient management, has been required, by the bankers, to agree to put aside from its net surplus such sums of money each year as will produce one-half of the amount necessary to provide additions and betterments by the time the total loan is due. This plan is wise financing from the bankers' viewpoint, but it demonstrates the needs of a carrier which arise in spite of the fact that it has so remodeled its property as to put it into the first rank as to average train loading. The money which produced the present high operating efficiency must now temporarily be denied a return, because sufficient rates of carriage have not been allowed the carrier.

Shall the Society, as an engineering body, attempt to formulate principles and methods of accounting which can apply only to the future? Is it to be expected that the problems involved in the valuation of public utilities will be solved by adopting any of the several theories advanced in the Committee's report?

As understood by the speaker, none of these theories deals with facts, but rather with assumptions of conditions which do not now exist, and

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can never be made to exist. All the value of all property cannot be completely measured with but one template in order to determine a proper return on capital. The term "value" is the *ignis fatuus* of the law; and, though it is defined more or less clearly in some few particular statutes of some States, it is generally left untrammeled, and free to wed with "reasonable", and, thus joined, comfort the Courts and economists, who, armed with these two indefinitive terms, are no longer in need of precise language.

The conclusions of this Society on subjects relating to engineering are generally held in high regard, and though the opinions of many of its members, on subjects relating to law, ecomomics, accounting, and the like, are respected by many, it seems unwise, at this time, in view of the Federal legislation and Court decisions which have been uttered since the appointment of this Special Committee, for the Society to go on record as acknowledging and sustaining any formulation of principles and methods for the valuation of railroads or other public utilities for the purpose of rate-making.

Mr. Lum.

D. W. Lum, M. Am. Soc. C. E. (by letter).—The Special Committee's report indicates a vast amount of labor and thought, and is very interesting as representing the personal views of the members of the Committee. The various features of the subject as treated, however, do not appear to harmonize with the practical requirements, making it necessary for the reader to choose between theory and fact in reaching a conclusion as to how to proceed in the great work that is before the country at this time.

The instructions of the Society to the Committee indicated that there was to be prepared a report to formulate principles and methods for the valuation of railroad property and other public utilities; the requirement being practically in line with that which later appeared in the provisions of the Federal Act.

As the Act was passed some months before the submission of the Committee's report, it would seem that the recommendations should conform to the requirements of the Law in order to be of practical service; and as great differences appear which cannot be reconciled, one must assume that there is error.

The writer notes with regret that the report, in its first lines, makes a most important deviation from the Act, as well as departure from the order assigning the duties of the Committee, in that it has assumed a physical inventory as the important measure of a rate-making value.

It appears proper to call attention to the fact that no reference to rate-making or earning allowance was made in the Act, nor in the order creating the Committee, and to limit the consideration in this way would defeat the object of the investigation. This view is practically expressed in the following statement by the Committee (page 67):

Mr. Lum.

"There are several reasons why a physical valuation, taken by itself, furnishes an unsatisfactory basis for determining rates, and other reasons why it does not seem equitable to omit other considerations".

If, therefore, this paragraph represents a concrete expression of the Committee's views, in order that the discussion may adhere to the practical duty assigned, it would appear necessary for the report to be revised by eliminating all reference to rates and substituting principles and methods for the valuation of railroad properties and other public utilities, as described in the Act, and as called for by the Society.

The proposed untried theory of allowed sinking funds and prescribed earnings assumes so many conditions which are in direct conflict with the facts developed in the operation of a property, that one is constrained to call attention to the very familiar example as a practical demonstration of the fundamental error in the report.

Two railroads, approximately parallel, and of substantially equal length, are competing: One, reporting in accord with the suggestions of the report, would inventory physical property at $20 000 000, the other, by the same rules, would inventory $10 000 000—the more expensive road, having steep grades and adverse conditions, may cost twice as much to operate. The Committee's theory assumes that the expensive road would and must charge double the freight and passenger rate for the service, which, of course, would direct all the business to the cheaper road. The result would be that the expensive road would get no business, and would "go to the wall".

Question: If the shipper insists on using the road offering the lesser rate, even though the Committee's theory requires that the more expensive road must earn just so much each year, how does the Straight-Line Method, or the Equal-Annual-Payment Method, or any method contribute any amount in dollars to the value of the road that is not patronized. Is it not true that such a road must conform to the requirements of competition, and, on the other hand, may not the road costing the lesser amount have a value far beyond the cost of its physical construction?

If, therefore, it is not practically possible to make a separate physical "valuation for the purpose of rate-making", it would seem that the Committee should adhere to the requirements of the Law, and develop the methods and principles that are necessary at this time.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS.*

By A. I. Stiles, Assoc. M. Am. Soc. C. E.

A. I. Stiles, Assoc. M. Am. Soc. C. E. (by letter).—The discussions of the report of this Committee, both in the *Proceedings* of the Society and in the technical press, seem to indicate a practically unanimous opinion that its figures and conclusions are far too optimistic. This view is supported by the writer's own experience. At the time that the information for this report was being collected, he was a member of an engineering organization employing about 20 men in the grade of transitman or higher, of whom two were members of the Society and reported their salaries to the Committee. These two men constituted the upper 10% of the entire force and their average salary was 204% of that of the remaining 90% of non-members. While somewhat more experienced than their fellows, it was by no means in such a proportion and it seems safe to assert that their salaries were at least 150% of what would have been attainable under similar conditions by the average engineer of equal length of practice. Mr. Stiles.

The writer heartily endorses the suggestion of Mr. Lundgren† that much valuable information might be obtained from a study of the salary lists of Government bureaus and large corporations, none of which seems to have any difficulty in obtaining plenty of qualified applicants for engineering positions at salaries much inferior to the Committee's averages. In the engineering department of one large railway, receiving hundreds of applications yearly, all college graduates without practical experience were informed that they would be required to begin as "track apprentices", at the same wages as common

* Continued from February, 1914, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., for February, 1914.

Mr. Stiles.

labor. No particular effort was made to instruct such men as accepted this offer. They were assigned to section gangs apparently at random, and sometimes acquired valuable experience in picking up wrecks, relaying rail, ballasting, etc.; in other cases they spent months in sweeping stations, cutting grass, picking up scrap, and similar tasks. A large proportion quit in disgust and at least 75% of the "graduates", expressed themselves as considering their apprenticeship as time lost. On the completion of the "course", having at least qualified for gameness, they were employed as assistants on surveying parties at initial salaries of $50 per month and field expenses, rising gradually from "stake artists" to levelmen and transitmen. It was the announced policy of the department to promote those who made good at the rate of $5 per month after each half year's service, and to employ experienced men from outside sources at equivalent rates, so that, after 5 years' service, an assistant engineer would receive $100 per month. About 75% of them never got any higher, and finally resigned. Division engineers were paid from $150 to $200 per month, or perhaps more in rare instances, and about 5% of the entire engineering force received salaries in excess of this rate, after from 10 to 20 years of experience. Many young engineers take their first few jobs almost entirely regardless of remuneration, and on account of the valuable experience which they expect to acquire. It is much to be regretted that in many instances such experience fails to materialize, and they find themselves in the position of having purposelessly cheapened their own services and those of their fellow workers.

The advertisements frequently seen in the technical press in which persons claiming to be engineers of some experience make such statements as, "salary no object", "will accept anything", "desire to obtain work without resorting to pick and shovel", etc., reveal the principal reason for the low salaries paid to engineers in subordinate positions. The attitude indicated by these advertisers should be considered as highly unethical and steps should be taken by the Society to prevent its continuance. No union man out of work would endeavor to obtain new employment by agreeing to accept less than the standard wages of his trade, and an engineer of 10 years' practice who is unable to obtain a suitable position at a fair salary, should take up some other line of work for which he is better qualified, rather than lower the remuneration of the entire profession by continuing on engineering work at the wages of unskilled labor. It must be admitted that it would be a very difficult matter to effect the change of attitude necessary to reach the root of this evil, which would require that all engineers receiving $200 per month or less should be organized somewhat on union lines, a proceeding that few seem to consider practicable. As a preliminary to the definite discussion of any such plan, it is suggested that the Committee would do well to obtain some first-

hand information from the less fortunate elements of the profession by answering a number of current advertisements for positions or even by inserting advertisements of its own, obtaining detailed information from all applicants and tabulating the results. Mr. Stiles.

The remarks of Mr. McCullough* in regard to the numerous engineering employment bureaus are of such importance as to merit much greater amplification. Theoretically, such institutions fill a legitimate field in investigating the past records of engineers and in furnishing competent men to employers on short notice. It is possible that there are individuals and corporations in need of engineering services who could obtain them in no other way so satisfactorily as through these agencies, but the writer's experience leads him to believe that *bona fide* cases of this kind must be comparatively rare. A new enterprise might secure a chief engineer from such a source but he, in turn, should have little or no difficulty in obtaining his staff from among his former professional associates and their acquaintances, reinforced, if need be, in the lower grades, by men obtained through advertisements or by recommended graduates of technical schools. Nevertheless, it appears that large numbers of men are constantly employed through agencies, chiefly on account of the fact that many employing engineers appear to have little time for, or interest in, this phase of their work, and often allow it to be handled and controlled by unscrupulous subordinates. The writer is inclined to believe that a considerable majority of the engineers employed through bureaus are, in one way or another, the victims of exploitation. The establishment of the proposed employment bureau by the Society and the pledging of all members to employ only men recommended by it, as suggested by Mr. McCullough, would certainly be of incalculable benefit to a great number of engineers.

As most registration fees are now made refundable in the event that no position is secured, the profit of a bureau depends entirely on placing men. The legitimate demand for such service being relatively limited, employees of bureaus are accustomed to increase their business by invoking the personal interest of the appointing power. This interest is usually secured by paying to the person influencing the appointment approximately one-third of the fee paid by the appointee to the bureau. Many men have accepted money from their appointees in this indirect manner, who would not think of blackmailing them directly. As far as known to the writer, there is no agency which does not grant commissions of this nature in return for services rendered by "insiders". Under such conditions, there is little prospect of efficiency in an engineering organization, every transaction being made with an eye to the fees obtainable rather than to the interests of either employer or employees. Each separation from the service means the creation

* *Proceedings*, Am. Soc. C. E., for February, 1914.

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of a new vacancy; each appointment is the opportunity for another fee; every competent man employed tends to block future operations; every man of ordinary incompetency is an asset, as there is always valid reason for his discharge; and all promotion or other fair treatment of employees becomes false doctrine. The writer has never known any organization where these influences were in full control, but he has become acquainted with several where they were very potent.

In the case of one large company doing business in South America, considerable difficulty was encountered in getting a sufficient number of engineers experienced in tropical work, and, consequently, the salaries offered were relatively high. Yet applicants for positions who addressed themselves to the chief engineer's office were answered by form letter about as follows:

"We are in receipt of your application for a position with this company and will probably need an engineer of your qualifications in the near future. We have a contract with the ——— Engineering Agency for supplying us with all the men we may need and if you will make application through them there is little doubt that you will shortly secure the position you desire."

Application to this agency consisted chiefly in signing an agreement to pay a fee averaging about 30% of the first month's salary.

A chief draftsman in charge of about 30 men made a regular practice of replacing at least three every month at a personal profit of about $30. No matter how poor a draftsman might be, he was never discharged until he had entirely paid his fee, and, should he give satisfaction, he might remain almost a year, but was never promoted, higher positions being filled by new men paying larger fees. On the other hand, no applicant in person ever got a position but was referred to the ——— Bureau, which was thought to know of work elsewhere.

On a large construction job in the United States, about ten inspectors were employed through agencies, and averaged about 6 months' tenure of office. The contractors were doing very poor work, apparently in collusion with the higher officials of the company. If an inspector was honest and insisted on work approximately up to specifications, he was promptly discharged for some triviality. If he was guilty of accepting poor work, it was not discovered for some months and then the contractor was paid in full for such work and the chief saved his responsibility by immediately discharging the man found to have been incompetent and employing another, thus earning another fee.

The writer believes that the general condition of the Engineering Profession can be improved only by the gradual attainment of the following:

(1) Elimination of the great number of incompetents who now constitute such a large proportion of the lower grades;

(2) Standardization of the salaries of engineers, beginning with the subordinate positions and going as high as possible; Mr. Stiles.

(3) Prevention of the exploitation of engineers by the establishment of an employment bureau by the Society, to replace present private organizations in this field.

If the engineers in the higher grades will assist their juniors in this manner, they will find that in so doing they have bettered their own condition.

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THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.

Discussion.*

By John W. Alvord, M. Am. Soc. C. E.†

Mr. Alvord.

John W. Alvord, M. Am. Soc. C. E. (by letter).—It is a matter of great interest that so many keen minds have devoted their close attention to the discussion of this paper. The subject is a rather abstruse one, but it is one that will have to be fully understood by engineers interested in appraisal work. It is, of course, impossible to review the many interesting points brought out in the discussion; only some general thoughts can be here set forth, suggested by the reasoning in certain typical discussions.

The writer is interested to note, from Mr. Burns' discussion, the fact that the California, Idaho, and Missouri Utility Commission Laws require that sinking funds shall be established. To this information may be added the fact that the Indiana Utility Law, not only requires that such funds be established, but permits the reinvestment of such funds in extensions to the property, providing, however, that the value of such extensions shall not be added to the value of the plant as a basis of fair return or purchase.

Since this paper was written, the Society's Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, has reported, and among its findings there is one (No. 14) in favor of deducting depreciation for the purpose of establishing fair rates. Later, the Committee issued an addition to the Progress Report, in which it amplifies methods of writing off depreciation, and concludes (page 18) "that any discussion

* Continued from February, 1914, *Proceedings*.
† Author's closure.

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of the subject which does not recognize the inter-relation between the method of providing for the depreciation and the method of valuing the property must necessarily be defective". Attention may be further directed to the fact that, since the paper was written, the Supreme Court, Appellate Division, First Department of the State of New York, in the Kings County Lighting Company *vs.* the Public Service Commission of the State of New York, First District, has upheld the Public Service Commission in its deduction of depreciation from reproduction-cost-new, reasoning out the matter in an opinion of several pages.

The writer is of the opinion, after a careful reading of the discussion and the Progress Report of the Committee on Valuation of December 1st, 1913, and its additional report, that what is needed at this time is a paper on fundamental principles of public utility valuation, and not papers dealing with details of rate-making and depreciation. The confusion of thought existing at this time appears to be caused by lack of understanding of the fundamental principles of utility valuation by a large number of engineers who are now interested in the question, and it will be hopeless to discuss details intelligently until a substantial agreement on such principles is had. As the fundamental principles of valuation work rest in the law and its interpretation by the Courts, many engineers apparently are not generally familiar with them. A brief effort will be made in this closure to clear up one or two of the more important ones which it is necessary to comprehend in order to read the discussions of this paper intelligently.

The writer would select for review, as typical, two of the discussions, one of which has apparently assented to the propositions contained in his paper, and the other of which has dissented therefrom. The first, by Mr. F. Lavis, points out (somewhat diffidently) what seems to him to be the difficulty with that class of valuers who dissent from the deduction of the depreciation from cost new.

To the writer's mind, Mr. Lavis strikes at the root of the matter. The difficulty clearly lies in the fact that the objectors do not take note of the fact that the appreciations of plants and property are included by the reproduction method, and must be considered in connection with the depreciations, and that, if the objectors would exclude depreciation, they must logically exclude the appreciation that the reproduction method usually introduces. The same arguments which one would use to exclude depreciation can be profitably used to show that questions of appreciation should not be entertained. This is fully stated in the paper.

It is a fair guess to say that failure to recognize this fundamental principle arises because some of the engineers now studying these questions for the first time have not yet had the opportunity to serve on actual appraisals, or, if so, they have not served on both sides of

the valuation problem, which is quite as important. This appears, to the writer at least, to explain the attitude of some of the more strenuous objectors to the theory here set forth. Mr. Alvord.

No one who has been in valuation work any length of time, and has served on both sides of the controversy, can fail to notice that there are three kinds of mental attitude among valuers:

1st. There are those who approach the subject from the standpoint of the protection of the rights of property and its conservation.

2d. There are those who approach the subject from the standpoint of the rights of the public and their interest in property devoted to public use.

3d. There are those, often recruited from the other two, who are influenced by the sincere desire to find the just and equitable relation between the public interest, on the one hand, and the private interest, on the other.

Now, in these matters we are, at the present time, really studying the third type of problem; that is, the just and proper relation between the public, on the one hand, and property, on the other, and not the protection of capital alone or the public alone.

We have found in the past, and undoubtedly shall continue to find in the future, that the public, in contested cases of valuation, is extremely keen, through its counsel and engineers, to ascertain and deduct every possible item of depreciation that can be taken away from a public utility property, and, on the other hand, that investors and owners, likewise through their counsel and engineers, are fully as keen to observe every fact and element which has appreciated their property. Both these two opposing tendencies are fundamentally sound, and, as usual, the truth lies in a careful conclusion which would allow to both all reasonable depreciation and obvious appreciation. When once the principle is grasped that both depreciation and appreciation must be taken into account, much of the trouble in dealing with this depreciation problem will have vanished. The Constitution of the United States gives the owner of a public utility property the right to have it valued "as of to-day," and as a "property", with all its accretions and growth in value, much of which may not be represented by any actual cash cost; and this right goes along with the right of the public not to pay a return on the full original cost of the property if it can be shown to have lessened in value in any way. This was stated clearly in the paper, but seems to have escaped the attention of some of those who have discussed the matter.

In the case of railroad properties, for instance, the appreciation of land values must certainly be a very large item, far outweighing any loss in value through minor depreciations of rails, cars, rolling stock, and the like. In the case of electric lighting properties, on the other hand, depreciations may often largely outweigh appreciations.

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The second discussion which the writer has selected as typical is that by Mr. Stuart K. Knox. His ideas have been set forth with great clearness and definiteness, and their amiable reasoning is so different from the unpersuasive tone adopted by Mr. Humphreys that one must be tempted, if possible, in the same spirit in which they are made, to note why one must differ from them.

Mr. Knox dissents from the writer's view on the second proposition, and introduces an interesting illustration (in amplification of the general statement in the paper), through which he seeks to show the absurdity of basing rates on depreciated property. He describes a water plant consisting solely of a pipe line, 10 miles long, costing $1 000 000, and having a life of 50 years, through which water, purchased at wholesale, is conveyed and sold at wholesale. A sinking fund of $6 550 is placed in the bank each year at 4% compound interest, which will amortize the entire line at the end of the assumed life. Mr. Knox reasons, as a first proposition, that if the sinking fund was kept intact as part of the property, the rates for service should be predicated on cost new, or, in other words, the accrued amount in the sinking fund should be permitted to earn at the same return as the rest of the property, and he concludes that this proposition, as set forth in the paper, appears to be "unassailable." The second proposition, however, he dissents from; that is, that if depreciation funds are detached or withdrawn from the property and used by the owner elsewhere in private gain, he cannot hope, as a matter of proper protection to the public, to receive rates which include a reserve fund which is not actually in hand, and so in hand that it is really part of the property. Mr. Knox proceeds to show that this proposition is not sound, by supposing that the owner of the pipe line does withdraw his sinking fund allowance and invests it elsewhere outside of the water property at the same rates of interest as it should earn as a sinking fund, and, further, supposing that the owner, in the tenth year of operation, becoming alarmed at the dwindling rates on his water transportation business, restores the fund, and christens it a "sinking fund", as part of the property.

This illustration is typical of a considerable number that have been introduced both in this discussion and in the report of the Committee on Valuation.

The difficulty of utilizing this kind of illustration is that not only is it quite unlike anything that happens in practically operated utilities, as has been set forth at length in the paper, but the further and more fundamental difficulty that it is not applicable to the problem here presented, because it is, pure and simple, the amortization of an original and actual cost. In valuation work, we are not considering the retracing of a past investment and its exact mathematical protec-

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tion, at all, but we are considering present-day valuations made on a live and growing property in its entirety.

So far as the writer's knowledge goes, it is not customary to treat the actual past cost as something from which depreciation should be deducted. It is true that the representatives of the public often make a plea for such a procedure, but Courts, commissions, or appraisal boards, with which the writer is familiar, have not decided that depreciation should be deducted from actual past cost for the purpose of making rates unless it can be shown that past cost is present value, nor does this paper even intimate the possibility of such a proposition.

What has been attempted to set forth is that in the reproduction method—that is, cost new as of to-day—if used for determining values, should have deducted from it the depreciation before using it for the computation of rates, if an adequate fund for depreciation is not in hand. A careful re-reading of the paper will make this clear. It is true that Mr. Grunsky calls it "past investment", but the writer only quotes Mr. Grunsky's exact language to make his own distinction, that it is "property now" that is valued, more conspicuous.

Now, Mr. Knox, in all his discussion, is evidently dealing with past cost, because in one place (page 216)* he says: "Assume that the reproduction-cost-new of the pipe is $1 000 000", and in the very next paragraph he says: "In the construction of this pipe line, the owner converted $1 000 000 of money into physical plant."

"Reproduction-cost-new" with him evidently means original cost, so far as his illustration and argument is concerned at least, for that idea runs through all of his succeeding reasoning on this illustration. On page 223* he says:

"On the other hand, if we neglect the fluctuations in the prices of labor and materials, reproduction-cost-new will remain an unvarying quantity as long as no additions are made to the property."

And, further:

"The reproduction-cost-new having once been ascertained may subsequently be kept up to date merely by adding the cost of extensions as these are made, and the 'adequate' annual depreciation reserve."

The writer has been actively engaged for the past 15 years in valuation work, and has never found yet that reproduction-cost-new would remain an unvarying quantity, even in plants which were not being extended. Nor can he understand how the value of a property, having once been ascertained, can subsequently be kept up by adding the cost of the extensions, for properties usually grow in value or decrease in value with the growth or decrease of the surrounding population and the general increase or decrease in opportunity to give service, and

* *Proceedings*, Am. Soc. C. E., for January, 1914.

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this entirely outside of and irrespective of the money actually put into them.

Mr. Knox must unquestionably realize, as any engineer would realize on second thought, that utility properties in actual life may appreciate or depreciate in value outside of their original cost, and that, if growth is found, it is conceded to be the property of the owner, by the Constitution of the United States, as interpreted by the Supreme Court in numerous cases.

Mr. Knox's discussion, therefore, has only been taken as typical of this neglect to take into account possible appreciation, but he is followed in this respect by no less than six out of the sixteen discussions. The writer will try to make this fundamental difference clear.

"Past cost" is commonly understood to indicate the actual monies originally expended for the upbuilding of a property. It is one of the elements to consider in reasoning from cost evidence to value, but the Courts generally hold that it is not a controlling element.

Past cost is seldom a good measure of value except in the case of quite recent expenditures. Past cost may have in it monies expended for obsolescent structures, but it contains none of the appreciations which commonly accrue to a property, and often accrue without actual expenditure. Because it has none of these appreciations, it is not proper to deduct from past cost the appreciations; and in actual practice it is not done unless some attempt is made to compute and add the appreciations as well.

Reproduction cost is the method of computing the cost to reproduce the "property" as of to-day, in a manner that is humanly possible, and at market prices for labor and material, and including the development of the business. Consequently, it contains many of the appreciations that have come to the property by reason of the growth of its environment. Not all of these appreciations have cost actual cash; some of them are natural accretions or earned and unearned increments, which have cost the owner nothing. From such an estimate should be taken the depreciations which the existing property has suffered by reason of its age, changing conditions, or use. After this is done, if the result, in connection with other lines of evidence, is found to be properly value, as distinguished from cost, then that value may be taken to estimate the fair return.

Now, all this is elementary, and in preparing the paper it was assumed that these fundamental principles were understood, but to guard fully against misconception at the very outset, the paper states that we are discussing only that line of evidence as to value known as the reproduction method, or cost-new-less-depreciation.

Mr. Wilgus speaks of a balance sheet showing an excess of assets over liabilities. This surely relates to past cost. Farther along he

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speaks of an "impaired investment", showing that he has investment in mind and not reproduction-cost-new.

Mr. Willoughby says: "it is the ultimate finding of the accounting * * * that fixes the value of the utility," and though he immediately apprehends that we are limited to reproduction methods, yet he later speaks of such methods as though they were recent past cost, and in his illustration lapses into past cost completely again.

Mr. Boes clearly does not have in mind that we are discussing a "property", the value of which is fixed as of to-day.

Mr. Vensano argues entirely from past cost, and uses past cost in his illustration.

The Special Committee on Valuation of Public Utilities argues entirely from original cost, and, in its illustrations, appears to be constantly thinking about the proper protection of original cost.

Now, taking Mr. Forbes' example, and conceding his assumptions; that is, that original cost is "value" continuously through the life of the plant, the writer would promptly concede the correctness of his argument, that depreciation should not be deducted in any case for the purpose of making rates. There can be no rational dissent from such a proposition.

Practically, however, past cost is almost never present value. A thing may be worth what it cost, or it may be worth much more, or it may be worth much less; hence, past cost is very rarely accepted by the Courts as the best evidence of present value.

Even reproduction cost, less depreciation, is not always value. A plant may often be worth more or may be worth less than it would cost to reproduce it, and though some utility properties depreciate more rapidly than they appreciate, as a usual thing they appreciate much more rapidly than they depreciate, and both facts must be taken into account. In other words, original cost, plus the algebraic sum of all the changes in cost and theoretical depreciations, should equal reproduction cost, less depreciation, as of to-day, if it were humanly possible to make such an analysis correctly, but even so, we will not have arrived at value; other matters have to be considered as well.

In fixing fair rates, the Courts have said, over and over again, that it is the fair value (not cost) of the property used and useful for the public that must be taken into account.

It is interesting to note that most of the water-works engineers who have had experience in appraisal work have not misapprehended the fundamental proposition here set forth. Mr. Kiersted and Mr. Burns, particularly, have thrown a great many interesting side-lights on the main question.

To emphasize the matter further, it may not be inappropriate to revert to Mr. Humphreys' illustration of the hen (page 227* of Mr.

* *Proceedings*, Am. Soc. C. E., for January, 1914.

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Knox's discussion). Like a good many illustrations that do not illustrate, this one is misleading, in that it is not comparable with the actual conditions as we find them, for to make the illustration comparable with ordinary utility problems we should imagine that the hen lays more eggs as time goes by than she did originally, and is at the same time growing old.

What is the value of this hen to the consumer? Clearly more than it originally was, modified by the fact that she might soon die, and whatever the philosophy of the hennery farm manager, the Courts and commissions and practical appraisers would take both facts into account in fixing the price of eggs, especially if there were only one hen.

In conclusion, the writer is of the opinion that the trend of public utility regulation in the future is going to lean more and more to the requirement, in municipal utilities at least, that funds for depreciation, renewal, and contingency shall be largely, if not entirely, kept in hand as part of the property, subject to the control and supervision of the commissions. Such funds are not necessarily idle, as has been suggested, nor are they withdrawn from useful activity, but the public is undoubtedly much better protected, and the property is stronger and the owners' credit better when such funds are within public control and supervision.

It is a significant fact that in several cases, which have come under notice of late years, where new financing of public utilities, not under commission control, has been required, that the bankers have actually insisted on a replacement fund being created, and constantly maintained to a required amount, in order to make the property more sound and the securities issued more secure.

The case of the steam railroads presents a somewhat different aspect from the ordinary municipal utility. So much of the property of the steam roads is in land, which does not usually depreciate, and the remaining property depreciates so rapidly that, as pointed out in the Third Avenue Railroad case by Mr. Floy, it may be difficult and undesirable to create a special fund for depreciation, or find any method that would be more simple than that of maintenance, pure and simple. This, however, should not preclude the deduction of depreciation from reproduction-cost-new in cases of valuation where appreciations had been duly considered.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES EDMUND CHILDS, M. Am. Soc. C. E.*

DIED JULY 16TH, 1912.

James Edmund Childs was born at Neversink, N. Y., on July 11th, 1848, and began his railway work, in April, 1865, with the Engineering Corps engaged in the location and construction of the New York and Oswego Midland Railroad, now a part of the New York, Ontario and Western Railway.

During 1869 and 1870, he was Assistant Engineer on the Chicago and Michigan Lake Shore Railroad, now a part of the Pere Marquette Line. Later in 1870 he was Resident Engineer of the Buffalo, New York and Philadelphia Railroad, now a part of the Pennsylvania System, and in the following year, 1871, Division Engineer of the Rochester and State Line Railroad, now a part of the Buffalo, Rochester and Pittsburgh Railroad.

In 1873 Mr. Childs was engaged in the relocation of a division of the Wisconsin Central Railroad, as Division Engineer, and, in 1874 and 1875, as Assistant Engineer in charge of some improvements on the New York and Harlem Railroad, now a part of the New York Central System.

In 1876, Mr. Childs returned to the Rochester and State Line Railroad as Chief Engineer and Superintendent, and five years later, in 1881, came back, as General Superintendent, to the re-organized New York and Oswego Midland, on which as a boy, sixteen years before, he had begun his railway work.

He was also Assistant General Superintendent of the New York, West Shore and Buffalo Railroad during its construction and at the time it was opened for traffic in 1883.

In February, 1886, he was made General Manager and, in September, 1904, Vice-President and Director, of the New York, Ontario and Western. Mr. Childs was in the service of this company continuously for more than thirty years, with the exception of one year, 1889, when he was offered and accepted the position of Assistant General Manager of the Lake Shore and Michigan Southern Railway properties.

In the following year, however, he returned to the service of the New York, Ontario and Western, at the earnest request of its President

* Memoir prepared from notes by William A. Haven, M. Am. Soc. C. E., by Edward Canfield, M. Am. Soc. C. E.

and Board of Directors, in order to forward and complete the Ontario, Carbondale and Scranton Railway, then under construction from the main line to the City of Scranton, through the upper anthracite coal region. Mr. Childs remained in the service of this company as General Manager until the time of his death, filling also the positions of Vice-President and Director. By his energy and devotion to its interests and his knowledge of the property and territory which it served, he did more than any one else to promote its welfare and increase its traffic. His long service with the officers and employees, and the intimate and friendly character of his relations with them, rendered his death not only a loss to the company, but a personal sorrow to his associates.

In 1882, Mr. Childs was married to Laura Grant, a daughter of the late William H. Grant, M. Am. Soc. C. E., who survives him.

Mr. Childs was elected a Member of the American Society of Civil Engineers on December 4th, 1878.

JOSEPH POTTER COTTON, M. Am. Soc. C. E.*

DIED DECEMBER 13TH, 1913.

Joseph Potter Cotton, a son of Isaac H. and Rhoda Lamont (Potter) Cotton, was born in Bowdoin, Me., on May 8th, 1837. He was descended from old New England stock. He received his education in the public schools and academies of his home district, and worked on the neighboring farms. Later, he taught in districts schools in Maine, New Jersey, and Pennsylvania.

While he was engaged in teaching at Easton, Pa., the Civil War broke out, and, in 1862, he served as Orderly Sergeant of a company of militia at that place. In 1863, he raised and took command of Company C, Forty-eighth Regiment, Pennsylvania Volunteers, which was mustered into the service of the United States.

In 1866, Captain Cotton began his engineering work as Rodman and Leveler on the survey for the Lake Superior and Lake St. Croix Railroad in Wisconsin. In September, 1867, he entered the Government service, under the late General G. K. Warren, Corps of Engineers, U. S. A., and was engaged as Assistant Engineer on surveys of various rivers and bridges in Minnesota, Mississippi, Wisconsin, and Ohio. In 1871, he went to Newport, R. I., and, under General Warren, had charge of the construction of the breakwater at Block Island. In 1872 and 1873, with the late E. S. Chesbrough, Past-

* Memoir prepared by the Secretary from information on file at the Society House.

President, Am. Soc. C. E., he made a survey of Newport, in connection with a plan for a new system of sewerage, which plan, slightly modified, was adopted and forms the basis of the present system. Captain Cotton continued in the service of the Government until 1883, being engaged on river, harbor, and fortification work. In 1882 and 1883, he served as Commissioner and Engineer on the Seekonk River Bridge, at Providence, R. I.

In 1883, he resigned to engage in the private practice of engineering. He made his home at Newport, and became actively interested in city affairs and public improvements. From 1876 to 1883, he served as a Member of the School Committee; he was also Overseer of the Poor for three years, and Street Commissioner in 1890 and 1891. He was one of the group of citizens who framed the present city charter, and, on its adoption, he became a member of the Representative Council, serving on several of its important committees. He continued as a member of the Council until his death.

Captain Cotton was also deeply interested in the social betterment of the city and its people. He was one of the founders of the Charity Organization in 1880, and from that time until his death was an active member of its Board of Reference. He assisted in establishing the Holly Tree Coffee Rooms, the Law and Order League, and the Industrial School, and was for some years a Trustee of the Newport Hospital. His greatest public service, and one in which he took the greatest interest and pride, was rendered in connection with the Newport Co-operative Society for Saving and Building, which he was largely instrumental in founding in 1888, and of which he was the first and only President. The growth, prosperity, and efficiency of this institution is owing more to him than to any other man.

On March 26th 1867, he was married to Miss Isabella Cole, who died in 1908. He is survived by two sons, Dr. Frederic J. Cotton, of Boston, Mass., and Joseph P. Cotton, Jr., of New York City.

Captain Cotton was actively engaged in work up to the very end of his long life. He died in his sleep on December 13th, 1914, without pain and without premonition of death.

A few days later, Rear-Admiral French E. Chadwick, U. S. N., retired, wrote of him:

"Our town feels that in the death of Captain Cotton it has lost a mainstay. This loss is not only in our being deprived of a wise counsellor, an upright, clear-thinking citizen working always for the good of his fellow-men, but we have lost an exemplar of character. To old men he was an example, in age, of activity in business and public life; to the younger he was a pattern, for he exemplified in very full degree what character in the large sense means."

Joseph Potter Cotton was elected a Member of the American Society of Civil Engineers on June 7th, 1876.

NED HERBERT JANVRIN, M. Am. Soc. C. E.*

DIED JULY 17TH, 1913.

Ned Herbert Janvrin was born at Somerville, Mass., on May 20th, 1871. He was the son of Hiram Gilmore and Catherine Marriott Plummer Janvrin. An ancestor was one Jean Janvrin of the Isle of Jersey, whose son, Captain John Janvrin, sailed from Lisbon, Portugal, in 1696, and settled in Portsmouth, N. H. John, a son of Captain John Janvrin, born in 1707, was graduated from Harvard College in 1728. William Janvrin, the great-great-grandfather of Ned Herbert Janvrin, married Abigail Adams, a niece of President John Adams. A maternal ancestor, Francis Plummer, emigrated from Wales, at the foot of the Snowden Mountains, in 1635, and with his wife and two sons settled on the bank of the Parker River in Newburyport. He was the first man to keep a tavern and operate a ferry in that part of Massachusetts.

Mr. Janvrin received his early education in the schools of Somerville, having been graduated from the High School in 1890. In the fall of that year he entered the Massachusetts Institute of Technology, from which he was graduated in 1894 with an S. B. degree. George F. Swain, Past-President, Am. Soc. C. E., writes of him that "he was a conscientious, faithful, and very capable student, lending his influence toward a proper standard of discipline and attainment, and he left the school with the respect and confidence of all his teachers."

Almost immediately after his graduation he entered the drafting-room of the Boston Bridge Works, serving there until May, 1895, under J. R. Worcester, M. Am. Soc. C. E., Chief Engineer, who was impressed with his brightness, quickness of comprehension, and accuracy.

From May to November, 1895, he was employed as a Traverseman with the United States Geological Survey on survey work then in progress in New York State. He then returned to the drafting-room of the Boston Bridge Works, remaining there until December, 1896. From that time until May, 1897, he was with Mr. Worcester, during which period the train-shed for the South Station, at Boston, was being designed. Mr. Janvrin did a great deal of responsible work in the way of calculating and checking, particularly in connection with the girder work of the midway floor. In this position he showed that he was developing as an all-around designer, retaining all he had learned by experience and earlier training.

From May, 1897, to July, 1899, he was employed in the Bridge and Construction Department of the Pennsylvania Steel Company, at Steel-

*Memoir prepared by Robert Ridgway, M. Am. Soc. C. E.

ton, Pa., as a Checker and as Engineer in charge of drafting on the designs of steel bridges and viaducts.

Between July, 1899, and March, 1900, Mr. Janvrin was Assistant Bridge Engineer for the Metropolitan Street Railway of Kansas City, Mo., designing several of the Company's bridges and viaducts. He then entered the service of Messrs. Waddell and Hedrick, of that place, remaining with them until August, 1900, during which time he was connected with the design and construction of the Kansas City Viaduct.

From September to December, 1900, he was again with Mr. Worcester, assisting in the design of the steelwork for the Boston Elevated Railway Company, and from January to March, 1901, he served with Norcross Brothers, Worcester, Mass., as a Draftsman on steel building construction.

In April, 1901, he went with the American Bridge Company, remaining with it until September 15th, 1905. Until August, 1901, he was a Draftsman in the Pencoyd Plant. From then until June, 1903, he was Draftsman and Assistant Engineer, respectively, in the Eastern Division drafting-room, then located at Pencoyd, Pa., and in the Eastern Division estimating-room, also at Pencoyd. From June, 1903, to the middle of September, 1905, he was employed as Assistant Engineer in the Erecting Department of the Pittsburgh Division. His work there was mainly in connection with contracts for which the Company had sublet the erection to others. Among such works may be mentioned the erection of the Louisville and Nashville Shops, at South Louisville, Ky., and the Norfolk and Western Bridge, at Portsmouth, Ohio. The notable qualities of his character, which were impressed on his superiors on this work, were his absolute honesty, his quiet, industrious habits, and his ability as a thoroughly competent field checker of detailed drawings.

From October, 1905, to April, 1906, Mr. Janvrin was with Herbert C. Keith, M. Am. Soc. C. E., Consulting Engineer, of New York City, as a designer on several heavy steel bridges for the New York, New Haven and Hartford Railroad. Here he showed a keen perception of the points involved in several special problems which were presented, and great ability in solving them.

On April 2d, 1906, he reported for duty as Assistant Engineer with the Board of Water Supply, City of New York, and remained in its service, engaged on designs and surveys for and the construction of the Catskill Water Supply System, until his death. His purpose in taking up this work was due largely to a desire to increase his knowledge in a branch of engineering different from that in which he had been so long engaged. During this period, as opportunities offered, he received several promotions in rank and pay, on the recommendations of his superiors, for efficient service. Until September, 1906, he

was assigned to the Reservoir Department, on general survey work for the Ashokan Reservoir, during which period he had charge of a stadia party on preliminary topographical surveys. He was then transferred to the Designing Division of Headquarters' Department, and in the following spring was placed in charge of the construction and testing of an experimental section of reinforced concrete pipe, 11 ft. in diameter and 210 ft. long. The experiment was made with a view of ascertaining whether this type of construction was feasible and desirable for some of the smaller siphons of the Catskill Aqueduct where the head to be provided for was not great. The pipe consisted of seven 30-ft. units, each unit differing from the others as to the mixture of concrete, the details of reinforcement, etc. Mr. Janvrin gave the details of this work his earnest and intelligent attention, and his report of it, and of the successful hydrostatic test which followed the construction, was of material assistance to the designing force and added much to the knowledge of the subject of reinforced concrete pipes. In December, 1907, he was transferred to the Northern Aqueduct Department and assigned to its Newburgh Division, in Orange and Ulster Counties, where he assisted in locating the cut and cover aqueduct on that Division of 15 miles. When the work was placed under contract, he was at first given charge of Contract No. 16, a section of aqueduct 2½ miles long, and, later, in June, 1909, of Contract No. 45, about 5 miles long. As the intensity of the work increased, Contract No. 45 was divided into two engineering sections, and on April 1st, 1910, Mr. Janvrin was given charge of the south section. A notable feature of his section was the building, in two places, of the large concrete aqueduct of 500 000 000 gal. daily capacity on a heavy foundation embankment of earth, where the surface of the ground fell below the elevation of the aqueduct invert. The larger of the two embankments was 900 ft. long, and its maximum height, from the surface of the ground to the bottom of the aqueduct structure, was about 18 ft. On the practical completion of this section, on May 1st, 1913, he was transferred to the Designing Division of Headquarters' Department, where he remained until his death. In this last period of his work, his thorough grounding in the principles of stresses and strains in structures and his complete knowledge of steel and concrete structures were of great assistance in establishing methods of design applicable to the many subsurface gate-chambers of the City Aqueduct tunnel.

While on a vacation he was stricken with spinal meningitis, from the effects of which he died at Boston, Mass., on July 17th, 1913. For nearly 20 years he had given his earnest attention to the active practice of his profession, and his death, in his forty-second year, ended a career of much promise.

A friend of Mr. Janvrin, who had known him from his boyhood, commenting on his characteristics and ideals, writes:

"The one thing that stands out is his single-mindedness. It has been my lot to know of no man whose life from beginning to end was so perfectly straightforward, whose character was more truly clear and clean, whose ideals were more high, but this singleness of mind stands out the most sharply. There was nothing complex in his make-up, and it seems to me in these days of mixed motives in the characters of the best of men oftentimes, that such perfect singleness of mind was as remarkable as it is rare. This same singleness extended to his intellectual processes. His really brilliant mind moved straight ahead always, without waywardness. In all his social relations, the same characteristic predominated. As a boy he was a generous, kindly companion and a helpful son. As a man he was upright and honorable with the world at large, staunch and loyal to his friends, unselfish and devoted toward all his family."

Another, who became associated with him in his work soon after his graduation from college, and who formed a friendship then which lasted as long as he lived, writes of him as

"A man of clean habits and high ambitions, of excellent ability, strictly honest and honorable in all his dealings, and true to his friends."

Those who knew Mr. Janvrin well recognize these traits, and will endorse the tributes to his character paid by these friends.

He was a Mason, a member of Temple Lodge 299, Kansas City, Mo., a member of the Technology Club of New York, and of the National Geographic Society. He was interested in outdoor sports and skilled in games.

He was married, on June 20th, 1907, to Miss Avis Genevieve Grimes, of Franconia, N. H., who survives him.

Mr. Janvrin was elected a Junior of the American Society of Civil Engineers, on October 5th, 1897; an Associate Member on June 5th, 1901; and a Member on April 4th, 1911.

DAVID NEILSON MELVIN, M. Am. Soc. C. E.*

DIED JANUARY 27TH, 1914.

David Neilson Melvin was born in Glasgow, Scotland, on July 21st, 1840. He was the son of David Melvin, of Paisley, Scotland, a successful card manufacturer at Oxford, England, and also a notable figure in the temperance movement in Great Britain.

* Memoir prepared by George A. Parker, Esq., Mech. Asst. to Supt., The American Linoleum Mfg. Co., Linoleumville, N. Y.

David Neilson Melvin received his early education at the Andersonian Institute in Glasgow, and, in 1855, was apprenticed to the firm of Crawhall and Campbell, Engineers, to study drafting and to work through the shops.

In 1861, on the completion of his apprenticeship, Mr. Melvin was employed as Draftsman and Mechanical Engineer by several firms in Glasgow and vicinity, particularly by Blake, Barclay and Company, for which company he designed and superintended the construction of machinery and fireproof buildings, for some of the largest sugar-refining mills in Great Britain. He also designed the machinery and buildings for sugar mills in the West Indies.

In July, 1863, he purchased an interest in a paper mill near Oxford, England, which he operated until the abolition of the British tariff on paper made the business unprofitable. In 1865, he went to Birmingham, England, as Assistant to Mr. Henry Lea, Civil Engineer, remaining in that position until April, 1867, when he came to the United States. Shortly after his arrival Mr. Melvin obtained a patent for an improved sectional safety steam boiler, and, later, for an automatic furnace door, and also for a high-pressure engine. He was also associated with Mr. T. A. Weston, the inventor of the differential chain pulley, in Buffalo, N. Y.

Later, he was employed as Engineer and Superintendent by the Rodgers Iron Manufacturing Company, of Muskegon, Mich., and A. F. Bartlett and Company, of East Saginaw, Mich., in the production of wood-working machinery and in the manufacture of steam engines. He also superintended the erection of some of the largest lumber mills in the Michigan lumber regions.

In February, 1873, The American Linoleum Manufacturing Company was organized, and Mr. Melvin was appointed its Engineer. With Mr. Frederick Walton, the inventor of linoleum, he designed and superintended the erection of the machinery and buildings of the Company's large plant at Linoleumville, on Staten Island, New York. On the completion of this work, he succeeded Mr. Walton as Superintendent.

In 1888, Mr. Melvin invented and patented the machinery for manufacturing an inlaid linoleum, and about 1900, he brought out a patent for wood inlaid. These goods are now being manufactured exclusively under his patents. He retained his position as Superintendent until his death, which occurred at Miami, Fla., on January 27th, 1914, after a lingering illness.

Mr. Melvin was one of the original members, and a Life Member, of the American Society of Mechanical Engineers. He was also a Member of the Richmond County Automobile Society.

David Neilson Melvin was elected a Member of the American Society of Civil Engineers, on July 3d, 1878.

PETER ALEXANDER PETERSON, M. Am. Soc. C. E.*

DIED NOVEMBER 21st, 1913.

Peter Alexander Peterson was born in Niagara Falls, Ont., Canada, on November 8th, 1839. In July, 1859, after a thorough education, he was articled as a pupil in Surveying to Thomas C. Keefer, Past-President, Am. Soc. C. E., who, at that time, was Chief Engineer of the Hamilton Water-Works and of the Hamilton and Port Dover Railway. For two years, Mr. Peterson was engaged on these works, with headquarters at Hamilton, and from 1861 to 1863, on a survey for a canal from Georgian Bay to Toronto, being stationed at the latter place. In July, 1863, he obtained a license as an Ontario Land Surveyor, and during the next two years was employed on various engineering works under Mr. Keefer.

In May, 1865, Mr. Peterson left Mr. Keefer's employ to engage in the private practice of engineering, and during 1865 and 1866, he had charge of the reconstruction of three large dams on the Grand River, to replace those which had been carried away by floods caused by ice jams.

In the summer of 1867 he made surveys, plans, and estimates for the Petrolia Branch of the Great Western Railway, and, in the autumn of the same year, was appointed Resident Engineer of the Northern Division of the New York and Oswego Midland Railway. In March, 1868, he was appointed Resident Engineer of the Bathurst Division of the Intercolonial Railway, in New Brunswick, which was then being constructed.

Mr. Peterson remained with the Intercolonial Railway until September, 1872, when he resigned to accept the position of Chief Engineer of the Toronto Water-Works, the construction of which, it was estimated, would cost $2 000 000, and included a filtering basin 3 000 ft. long, 10 000 ft. of conduit from basin to pumping well (4 500 ft. of which was a 36-in. flexible pipe laid across Toronto Harbor), pumping engines, reservoir, and more than 100 miles of distribution pipes.

In 1875, Mr. Peterson was appointed by the Quebec Government as Chief Engineer of the Montreal and Ottawa Section of the Quebec, Montreal, Ottawa and Occidental Railway, which, except for that portion between Terrebonne and Montreal, had been constructed between Quebec and Ottawa. The location of the unconstructed portion was under discussion and Mr. Peterson strongly urged the adoption of the direct line from a point near Berthier to Montreal *via* Bout de l'Isle, including a large bridge at the latter place. In July, 1878, however, the late Mr. Walter Shanly made a report to the Government of Quebec

* Memoir prepared by H. Irwin, Esq., Cons. Right-of-Way and Lease Agent, Canadian Pacific Railway Company, Montreal, Que., Canada.

in which he advocated the line finally adopted, namely, *via* Terrebonne, St. Vincent de Paul, and St. Martin's Junction, with two large bridges and two long, heavy grades up to and down from Mile End. Mr. Peterson's route was afterward adopted by the Canadian Northern Quebec Railway.

As an engineer, Mr. Peterson excelled in the location and building of the substructures of bridges, and as Chief Engineer of the Montreal and Ottawa Section of the Quebec, Montreal, Ottawa and Occidental Railway, he had charge of the construction of the Chaudière Bridge between Hull and Ottawa, having one span of 254 ft., one of 160 ft., ten of 150 ft., and one of 135 ft., and of several other bridges over the Rouge, North Nation, Lievres, and Gatineau Rivers.

In 1881, he was appointed Chief Engineer of the Atlantic and North West Railway Company, under the charter of which the Canadian Pacific Railway Company built the line from Mile End Station, near Montreal, to connect with the International Railway of Canada at Lennoxville. At the same time, but under another charter, this road was extended from the International Boundary, near Megantic, to Mattawamkeag, Me. While in this position, Mr. Peterson superintended the construction of the St. Lawrence Bridge at Caughnawaga, on this line, the piers of which, with the necessary lengthening, were strong enough to carry the heavy double-track steelwork which, by a strange coincidence, was completed only a month before his death. On this line were also several large steel bridges and trestles, including the bridge over the Richelieu River and high trestles at Ship Pond and Wilson Stream.

Mr. Peterson was also Chief Engineer of the Sault Ste. Marie Bridge and of the Mission River Bridge, in British Columbia, and of many other important works.

In 1890, he was appointed Chief Engineer of the Canadian Pacific Railway Company, which position he held until February, 1902, when he was obliged to resign on account of ill health. He then became the Consulting Engineer of the Company.

In August, 1903, he was appointed Chief Engineer of the Guelph and Goderich Railway then being constructed by the Canadian Pacific Railway Company, and held that position until 1908 when he was obliged to retire from active work owing to his failing health. He made his home in Montreal, Que., Canada, where he died on November 21st, 1913.

He was elected a Member of the Institution of Civil Engineers of Great Britain on December 1st, 1874. He was also a Charter Member of the Canadian Society of Civil Engineers, and served as Vice-President in 1889, 1892, and 1893, and as President in 1894.

Mr. Peterson had taken a leading part in railway construction in Canada during the latter part of the nineteenth century. He was

most conscientious in regard to the performance of his engineering work, and strict, but fair, in his dealings with others. He was very kind-hearted, and always willing to aid any of his staff, although he required from them close attention in their work and whole-hearted discharge of their duty to their employers. Being of rather nervous disposition, he was, at first, sometimes thought to be slightly abrupt in manner, but his unvarying courtesy soon removed that impression.

Peter Alexander Peterson was elected a Member of the American Society of Civil Engineers on January 5th, 1876. He served as a Director in 1892 and 1893, and as a Vice-President in 1896 and 1897.

WALLACE BERKLEY RIEGNER, M. Am. Soc. C. E.*

DIED JANUARY 19TH, 1914.

Wallace Berkley Riegner, the son of Aaron H. and Caroline S. Riegner, was born in Strawsburg, Franklin County, Pa., on January 27th, 1854. In his early boyhood his parents moved to Chambersburg, Pa., where he attended the public schools, from which, as well as from the Chambersburg Academy, he was graduated. He afterward entered Lafayette College from which he was graduated with high honors in June, 1877. While at Lafayette he received the Junior Mathematical Prize, the Senior Astronomical Prize, and was chosen to deliver the Honorary Philosophical Oration. He also represented his College in the Intercollegiate Mathematical Contest.

After his graduation, Mr. Riegner taught mechanical drawing in the public schools of Reading, Pa., for about one year. On October 27th, 1879, he entered the Engineering Department of the Schuylkill Canal where he remained until March 1st, 1880, when he went to Pottstown, Pa., as Assistant Engineer on the Philadelphia and Reading Railroad, remaining in this position until August 31st of the same year.

He then went South and from December, 1880, to December, 1881, was engaged on surveys and construction work, on the Elizabeth City and Norfolk Railroad. He afterward returned to the Engineering Department of the Schuylkill Canal, with headquarters at Reading, Pa.

In April, 1882, Mr. Riegner resigned his position with the Schuylkill Canal Company to enter the employ of the Philadelphia and Reading Railway Company. From April, 1882, to July, 1883, he was engaged as Division Engineer on the Shamokin, Sunbury and Lewisburg Division, and from July, 1883, to April, 1887, as Assistant Engineer on general field and office work and the design of railroad structures at the company's office in Philadelphia, Pa. In April, 1887, he

* Memoir prepared by William Hunter, M. Am. Soc. C. E.

was appointed Engineer of Bridges, which position he held at the time of his death which took place at Chambersburg, Pa., on January 19th, 1914.

Mr. Riegner was a Member of the American Society for Testing Materials, the Franklin Institute, and the Engineers' Club of Philadelphia.

He was an intelligent, energetic, and capable engineer, modest to a degree, with a strong grasp of details which enabled him to prosecute his work most successfully.

Mr. Riegner was elected a Member of the American Society of Civil Engineers on September 7th, 1904.

HENRY HELM CLAYTON, Jun. Am. Soc. C. E.*

DIED APRIL 8TH, 1913.

Henry Helm Clayton, the son of Lillie Sale and John Benjamin Clayton, was born in Sappington, Mo., on February 8th, 1885. He received his education at the Kirkwood Public School and the Manual Training School at St. Louis, Mo., and was graduated from the latter in January, 1903.

Mr. Clayton began work as Rodman with the Terminal Railroad Association of St. Louis, where he remained until June, 1903, when he accepted a position on the International Mexican Railroad. Having won a scholarship in the Engineering Department of Washington University, he returned to the United States in September, 1903, to enter that University, and in June, 1907, he was graduated with the degree of B. S. in Civil Engineering.

He then went to Texas as Rodman on construction work for the Rock Island Railroad, but during the same year he accepted a position as Computer for the Board of Examination and Survey of the Mississisippi River Commission. He remained in this position only a short time, returning to Texas to enter again the employ of the Rock Island Railroad Company, which position he retained until the work there was completed.

In January, 1908, Mr. Clayton formed a partnership for the private practice of engineering, with offices at Clayton and Kirkwood, Mo., which partnership was dissolved in December of the same year. In the meantime he had been appointed City Engineer of Wellston, Mo., but resigned this position to enter the service of the United Railways Company of St. Louis.

* Memoir prepared by John B. Clayton, Jr., Esq.

In May, 1909, Mr. Clayton joined the forces of the Missouri Pacific Railroad and continued in the employ of that company in various positions until his death on April 8th, 1913. In January, 1910, he was appointed Assistant Division Engineer on the Arkansas Division of the St. Louis, Iron Mountain and Southern Railroad. On June 1st, 1911, he was sent to Chester, Ill., as Assistant Division Engineer on the Illinois Division, where he remained until December 1st, 1912, when he returned to the Southern District.

Mr. Clayton was a man of irreproachable character and of strong personal magnetism, and was unusually energetic in the faithful execution of his work. Having a warm heart and a generous disposition, he was true and faithful as a friend. His make-up was that of a big man, and it is believed that he would have accomplished big things had his life been spared.

Mr. Clayton was a Thirty-second Degree Mason. He was elected a Junior of the American Society of Civil Engineers on December 3d, 1907.

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"CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION." W. C. HAMMATT. (To be presented May 20th, 1914.)

"HUACAL DAM, SONORA, MEXICO." H. HAWGOOD. (To be presented May 20th, 1914.)

"REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS." HARRISON S. TAFT. (To be presented May 20th, 1914.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

By Clarence T. Johnston, M. Am. Soc. C. E.

To be Presented September 2d, 1914.

Synopsis.

The laws generally applied in the United States in the administration of streams originated in the countries of Northern Europe. These countries developed under the feudal system, in contrast to Egypt, India, and China where, at a very early date, wise principles relating to the administration of streams were discovered and applied in practice.

Northern Italy is a striking example in so far as the development of principles relating to the administration of streams is concerned. Under the dominion of the Roman Empire, wise principles were applied. Spain brought other ideas of a beneficial character. The influence of the countries of Northern Europe, from which we borrow many of our laws and customs, was detrimental to the people of the Valley of the Po.

The doctrine of riparian rights is of feudal origin. It contains no positive principle, and it has outlived its usefulness. It can only be applied by the Courts, and the decisions of our tribunals of justice are

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

not in harmony as to the interpretations which should be given the doctrine according to character of uses and varying climatic conditions. We should not depart from a doctrine which has been followed for many years, until we have something better to suggest in its place.

The first principle that should be recognized is that of public ownership of streams, lakes, and other bodies of surface water. The recognition of this principle lays the foundation for an administration which, without great expense to the various water users, can protect all in proportion to the uses perfected, the character of such use, and with due consideration of all public interests.

Priority in time of use shall give the better right.

The right to use water gives the user no property interest in the water itself, and such right cannot be separated from the use, or transferred to other uses by such user. As long as the user has no personal control of the water supply, he has nothing with which he can speculate to the injury of others.

Preferred uses should be stated in order, in the law, and then protected in proportion to the value of each in preserving the lives of the people and promoting the general prosperity of any particular area.

Beneficial use should be the measure, the limit, and the basis of the right.

A declaration of abandonment should be made when the user fails to apply the water to some beneficial purpose within a reasonably short, specific time.

Even had these principles been recognized by the Courts, it would have been difficult for these tribunals to secure and judge the relative importance of the technical data that must be at hand before a just determination of rights may be made.

A compromise between Court and direct administrative control brings but little reform, because the responsibility is then divided and the defects of both systems are likely to be magnified.

The administration of streams lies within the field of the engineer. The best governed countries of the world, in this respect, have long since provided technical departments which administer this kind of public business in a very satisfactory way. The history of some of the countries of Europe is being repeated in America, and the engineer should be ready to accept the responsibility that, sooner or later, must come to him.

The manners and customs of the countries of Northern Europe have been introduced in the United States and accepted quite generally by the people. The climate and topography of the eastern half of the American Continent compare favorably with those of England, France, and Germany. It is natural, possibly, that the rules applied in the administration of streams, during the early history of America should be similar to the regulations enforced on the Thames, the Seine, and the Rhine. Where irrigation is essential to agriculture, we find an early interest manifested in problems relating to the diversion and use of water. The influences of the countries of Northern Europe in this particular would be more healthful if irrigation were practised there. It is probably unfortunate that the people of the United States have not been taught, by some early experience, that streams should be considered public property and one of the great natural resources.

In the older, irrigated districts of the world we find some good examples of wise laws relating to the administration of streams. China and India can teach us many lessons. The modern system adopted by Australia merits our study. Egypt, that land which has never enjoyed freedom, and has always been a victim of war, famine, and pestilence, administers its one river, the Nile, in a manner which can only call for praise from citizens of countries which boast of a higher type of civilization. Although the rulers of Egypt, both foreign and domestic, have robbed the people of their lands, have taxed them to the extreme limit, have compelled them to labor for the public without compensation, and by centuries of oppression have destroyed their initiative and broken their spirit, yet the Nile has always been public property. It is the great highway of commerce. It is the life of agriculture and the foundation of every other industry. The Nile is administered by public officers for the general benefit of the people. No private rights, which tend to threaten the prosperity of the people, are recognized or protected. The principles that have been accepted by Egypt are fundamental in their character, and they are applied uniformly throughout the extent of the Nile Valley. There is no conflict between farmers under the same canal, between communities depending on different canals, or between adjoining provinces. The principles that are essential are so simple that the native can understand and appreciate them. This immediately brings about a feeling of mutual confidence and respect between the water user and the officer in charge

of the local administration. Finally, there is no litigation in Egypt between users of water, whether they be individual farmers, corporations, or municipalities.

The laws of Imperial Rome, relating to the administration of streams, furnish examples which are superior to those we have introduced from the countries of Northern Europe. These laws were based on the idea of public ownership of streams. They recognized that water must be diverted from the natural channel of streams, in many cases, if beneficial uses are to be served and protected. They gave the builders of canals and other works the right to condemn private property. Those building improvements of this kind were obliged to respect the rights of others and to compensate those whose property was injured.

Northern Italy, like Egypt, has been a battle ground where the nations of the world have staged many important campaigns. Spain, France, and other nations have in turn dominated portions of the Valley of the Po. The influence of these invaders has been felt long after they surrendered political control. It would be assumed that the influence of Spain would not be detrimental in so far as the administration of streams was concerned. This was demonstrated. That country had gone through an experience that was profitable. The invasion of the Moors was beneficial to Spain in this respect, because these people from Northern Africa had been reared in districts where public control of the water supply was essential to life. The Spanish influence in Northern Italy, therefore, was salutary.

The influence of the people of France and Germany was detrimental. The countries of Northern Europe had developed under the feudal system. Because it was not necessary to divert large volumes of water from the streams to support the population, safe and sane principles were not evolved at home, and no apparent effort was made to discover and introduce rules that had been applied in countries which had experienced healthy, early development. The feudal countries always emphasized the importance of protecting private property for the benefit of the few owners thereof. The countries of Southern Europe and Northern Africa, on the other hand, recognized at an early date the necessity of public supervision of streams for the benefit of the many. When a portion of the Valley of the Po fell under the dominion of France, principles of feudal origin were intro-

duced. Concessions were made by the State and then by the Church. The fortunate individuals who received these concessions became owners of specific volumes of water, and speculation became general. The principles of the original Roman law were broken down here and there, and much confusion resulted. Litigation soon began, and many of these cases remained in the Courts in some form for several hundreds of years. Not until 1884 did Italy recover from this influence, in so far as its laws were concerned, and it will be many years before it recovers fully. Some plan must be formulated, and then carried into effect, whereby the old concession may be canceled. While Northern Italy was throwing off the customs it had inherited through early years of feudal association, some of our Western States were passing through a similar experience. The reforms that have taken place there and here are quite similar, yet neither country has profited by the experience of the other.

As the customs of Northern Europe, rather than those of other countries, were first introduced into the United States, we are somewhat concerned in their origin, the principles they embraced, and the method in which they were applied. As streams were of only passing importance to the feudal lords, and as private property was not held by the masses, it was probably apparent to them that a single rule might answer for all time. It was plain that some rule should be applied which would prevent the pollution of streams. As streams were of some importance to navigation, and as the owners of land bordering the watercourses desired stable conditions, in so far as discharge was concerned, it did not seem wise to encourage diversions from the rivers which would materially affect the flow thereof. The rule which gradually developed is approximately as follows: All persons owning lands abutting on a natural stream have the right to demand that the waters of that stream shall pass their lands undefiled in quality and undiminished in quantity. We call this the doctrine of riparian rights.

If this rule is carried into effect rigidly, water cannot be diverted from a stream in any way. The thirsty laborer who drinks from a convenient brook must diminish the "quantity" of water flowing therein. Streams must become more or less "defiled" with every contact with animal life. It was a theoretical rule, in the first place, this doctrine of riparian rights. As it had its birth in oppression and

despotism, so its application to the streams of our time has brought political and industrial slavery and injustice. The doctrine is a part of the unwritten law of England. Because it is theoretical in character and based on no well-defined working principles, it has furnished but little light to the Courts of our day.

The doctrine was introduced in the United States with the common law, and applied by our Courts, not by our law-makers. Its application has extended by judicial decree, so that its interpretations are only limited by the number of our Courts. It is seldom recognized and never defined by State or Federal statute. It was unjust to the Courts to thrust upon them the responsibility of protecting water users. When the law does not provide an administrative department for the purpose of transacting public business, those who would otherwise be protected must war among themselves and finally resort to tribunals of justice for relief.

We need no evidence, other than that contributed by the Courts themselves, to prove that our present methods of stream control, based on the doctrine of riparian rights, are not supported by principles that are sound. Every city, every water-power plant, every individual who resorts to the stream for domestic water supply, is violating the doctrine of riparian rights, yet this doctrine prevails almost alone east of the Missouri River. To be sure, the Courts hold that if the doctrine of riparian rights has been violated for a term of years the user may obtain a specific volume of water under what they call a prescriptive right.

Although this may give relief, in certain cases, yet it goes to demonstrate the weakness of the parent doctrine. It is an indication that the Courts recognize that some other rule must apply, if very obvious injustice is to be provided against.

The early decisions of the Supreme Court of Michigan hold that an owner of land abutting on a watercourse also owns the land of the bed of the channel to the "thread" of the stream. This term "thread" may be defined in theory, but presents some difficulties in practice. The Supreme Court of Michigan intimated, at first, that this was to be a general rule. It was to apply, and possibly has applied, to navigable streams as well as to those that are not navigable. Under this theory, the citizens of Detroit own the bed of the Detroit River to the "thread" of the stream. The same theory was stretched a little when

applied to the riparian lands bordering Lake St. Clair. It applied again to the St. Clair River, but when Lake Huron was reached the Court finally had to admit that the riparian owner had to limit his possessions at the meander survey of the lake. This would seem to show that a fundamental error had been made in the early decisions. The meander line should be considered the boundary between private and public property throughout. No private citizen can exercise exclusive control of the bed of a river without interfering with public interests. The doctrine of riparian rights, without modification, has but little to do with the protection of legitimate uses or the rights of the people at large. The Court, having no other rule to govern its acts, was compelled to construct its decisions with the timber at hand.

The writer has studied the application of the doctrine of riparian rights in countries of three continents. He has followed the history of its introduction into the United States, and watched its application as settlement has taken place, and as some kind of stream control has become necessary. He has found no two authorities who entertain the same views as to the theory of the doctrine, or as to its limitations in practice. If the doctrine has any value, if there are reasons why land owners along streams should have control of such streams, engineers, by this time, must have satisfied themselves as to the justification therefor. A discussion of the doctrine by engineers may be considered by some as an invasion of the field of the attorney. This is not true. It is not necessary to attempt to understand the intricacies of the theories that have been developed by the Courts. As the doctrine has been applied in the United States for more than 100 years, it should have demonstrated something as to its worth, and if it embraces a principle, this should be manifest to the layman. If the principles on which it is based are so obscure that they can only be understood and appreciated by the judiciary, then the doctrine should be abrogated, and something more tangible should be substituted in its place.

If the doctrine, such as it is, merits a stay of sentence, the engineer, who must always stand between the Court and the water user, should demand that its meaning be accurately and concisely defined, and its limitations be absolutely established.

The doctrine of riparian rights does intimate that the public (somewhat limited) is concerned in every stream. Court decisions, based

thereon, generally concur in the principle that the public owns the water, and only the use is acquired by private parties. The doctrine of riparian rights attaches the use to the lands owned by the riparian proprietor. A complete revolution is not necessary when we permit actual diversions of water from streams and place all rivers and their tributaries under the supervision of public officers. We simply pass from the application of a doctrine of negative principles to something positive and tangible. We may go back and study the application of the doctrine of riparian rights in other countries and satisfy ourselves as to the wisdom of affording it continued support here. When Great Britain enters new territory she does not apply the common law, to the detriment of her subjects. The doctrine of riparian rights does not apply in Egypt, South Africa, India, or Australia, as it does here. There, the Courts have never been compelled to make laws governing any resource.

Fortunately, we have some examples of better things, even in the United States. A few of our Western States have provided engineering administrations which, not only have custody of the records relating to the initiation of rights, but also determine the relative rights of all claimants. In order that such administration may do justice to private and public rights, principles are defined in the statutes that govern these departments. There, a land owner has no rights in a stream simply because his property is located along that stream. The great aim of the law is to encourage uses which develop the country, which establish homes, and, at the same time, preserve all public interests.

Before we discard a rule that has been applied for many years, we should have something better to substitute in its place. The administration of streams is very complex and difficult unless the few, wise, fundamental principles, discovered and enforced many years ago, are recognized. Though not assuming to refer to all principles that might apply, the writer will discuss a few of the more essential rules somewhat in the order of their importance. As has been intimated, the principle of public ownership of streams is the first essential to a just administration of the water supply.

a.—All streams, lakes, and other bodies of water, within the exterior boundaries of any country, should be, and always remain, public property. This means that the stream bed and the water flowing therein should be thus considered. There are many reasons why this prin-

ciple should be upheld. There is a physical reason, which is seldom considered. Streams are naturally of a public character, in so far as they are distinct from political lines. A stream, therefore, is a matter of general public concern. In addition, water cannot be privately owned. If Thothmes I had upheld the theory that private ownership of water is possible, and, to illustrate his ideas, had bottled a quantity and dedicated it to the generations of his family that followed him, the receptacle only would remain to those representing his house at the present time. Water is constantly moving. Like air, it is controlled by laws which were framed by a Higher Authority. We may enjoy the use, but not the possession.

As soon as streams, or any surface waters, become commodities for trade and speculation, the local public, responsible for such a condition, is openly defying natural laws and thereby imposing burdens on those who cannot protect themselves. No act of a despotic or careless government will work greater injury to its citizens than to permit, by decree or sanction of law, the private ownership of water. One might conceive of a government which would encourage a private monopoly of the air. It is at once apparent that this would be disastrous. Water falls in the same category, but because a profitable traffic can be carried on with apparent success, for a limited time, we are not shown, at once, the iniquity thereof. Though private parties have been able, frequently, to speculate in water, yet it is probable that the public has lost none of its inherent rights, and that both the buyer and seller have been deceived as to the real effect of the deal when consummated.

Public control of streams results in the determination of claims without great cost to water users. Where the Courts have jurisdiction, only the claimants who are financially able to maintain themselves in a controversy (that generally continues for years) can take an active part in the "adjudication" proceedings. Under the best administrative systems, the water user has no expense except that incurred in the payment of a nominal fee of two or three dollars. The public makes the surveys and measurements required, and collects all field data. The tabulation of rights is prepared by public officers, and hearings are provided where all users interested may protest against statements made as to the time water was used for beneficial purposes, and similar testimony. When the adjudication is accomplished, the final order is carried into effect by the same administration. All water users, con-

cerned in diversions from any stream and its tributaries, are brought together in the single proceeding, and no one, because of financial ability, or personal or political influence, has an advantage. High-class, public service of this kind is being performed in several of our Western States. The older administrations have had an experience of more than 20 years, and we can say now, with some assurance, that these are, in every way, a success. There, we find constitutional and statutory provisions which declare the waters of the States to be public, or the property of the State. These declarations relate to principles, and not to doctrines that have unlimited elasticity in interpretation.

b.—Priority in time of use shall give the better right. That is, the first user of the waters of a stream has a better right than any subsequent user, and the second user has a right superior to the third and those who follow, and so on. It is, perhaps, unnecessary for this principle to be discussed. It is absolutely necessary to make some such rule, particularly as between a large number of claimants concerned in a use of the same kind. It is manifest to one who gives the matter due consideration that, after the early users have made improvements and perhaps established homes, they should be protected as against later comers, who know what has taken place at the time of their appearance. This principle appeals with more force to an administrative officer, perhaps, than it does to one who has never participated in the practical work that falls to an engineer under such a system.

c.—The right to use water gives the user no property right in the water itself; such right belongs to the use and not to the user. This principle recognizes perpetual public ownership of water. The user is protected thereby, because he then belongs to a large class, all of whom are placed on the same basis, and no one person can impose on his neighbors. As long as the user has no personal control of the water supply, he has nothing with which he can speculate, to the injury of others. Although circumscribed by rigid restrictions, the legitimate user is protected, in so far as his use justifies protection, and, at the same time, he is given no special privileges.

Certain engineers framed a water law for one Western State some 20 years ago. They tried to have the measure specific in attaching the rights to use water for irrigation purposes to the land reclaimed, and to make it impossible for the owner of the land to dispose of such water right, except with the land. After some active campaigning

the bill was presented, and the framers were confident that this principle had been incorporated. They had a part in the administration of the law, and this principle was carried into effect. Within a short time an irrigator attempted to make a transfer of his water right separate from his land. Litigation began at once. This controversy in the Court was continued in an active manner for 10 or 12 years. At the end of that time the litigants had lost their farms, these having been consumed in Court costs and attorneys' fees. Finally, the Court decided that the law did not embrace the principle, and the transfer was thereby sanctioned. Those interested directly and indirectly in speculation in water joined in open hostility against any measure which had for its object the correction of the original law for the purpose of incorporating the principle therein. The friends of reform carried on a campaign for 6 years, and their efforts were finally rewarded by the enactment of adequate legislation.

To those who have observed problems of stream control from some distance, this principle may seem to be of but little importance. Wherever it has not been carried into effect extensive litigation has resulted and great injury has been done to water users who were not in a position to protect themselves. The argument made by those who oppose the principle is that any one who is damaged can secure redress in the Courts. This plan has but little to recommend it, because the weaker element is immediately eliminated from the contest. Further, the effect of a transfer, separate from the original use, is seldom apparent at the time of the transaction, and before those who are injured can appreciate the resulting damage and take steps to protect themselves, much money may have been spent in perfecting the new use. There is one exception to this general rule, however. It is often necessary to transfer rights to preferred uses. This should always be accomplished under public supervision. Should a city need water that had been formerly used for irrigation, power, or for other less important purposes, a procedure should be prescribed whereby a transfer of right to use (not a right to the water itself) may be made to the municipality in a public manner, so that all concerned may make their objections at the time, and, if injury results that cannot be foreseen, compensation can be made therefor under the direction of the administrative officers. Water cannot be maintained as public property when private parties sell water rights separate from the use to which they have been

dedicated, except under a public procedure and under the supervision and advice of public officers, so that, in carrying this last principle into effect, we are upholding another of equal importance.

d.—From the early records we are able to read it would seem that there has always smouldered in the breast of man, a feeling that all rights to use water cannot be classified in the same list. The first use that comes to mind relates to the domestic supply—water for drinking and household purposes. We must protect such uses, regardless of all other claims. It is plain that there should be preferred uses. There may be some room for debate as to the exact order in which these should be guaranteed protection. Climatic conditions, the nature of the water supply, and many other items enter as factors in such an undertaking, but an approximate estimate, which is of general application, can be made. No one will deny the necessity for protecting the individual supply for domestic purposes first. This is almost identical with municipal uses. Because many people, combined within the limits of a town, require more water than does any single claimant, the people of towns are often imposed on. Generally, under the doctrine of riparian rights, a farmer or other land owner is justified in taking water from a stream, adjoining his lands, for domestic purposes. As a rule, no effort is made to limit this use. We are compelled to suppose that it must be "reasonable". There seems to be no reason to presume that a single family, within the limits of a town, requires more water than does a farmer, living beyond its borders. Bearing in mind that the uses in both cases are equally high, and the only difference is one of local government, the question arises: When shall we deny the user within the town the same rights and privileges that are granted to the farmer? The Courts would probably justify a decision which discriminates between the two uses by the fact that the farmer is an owner of riparian lands and the city user is not. Assume that each user within the limits of the town is given a deed to a narrow strip of land connecting his property with the river bank. This will be an instance where conditions within the town may be made to fit a fine-haired theory. What, then, will be the status of the city user? He is a riparian owner in fact and a preferred user. Because he and his neighbors join together, and by combined efforts build a water system that will supply all within the limits of the city, there is no reason that we should assume that the character of the use has changed in any

way. The total demand on the stream may increase because of the growth of the town. The character of the use remains the same. Regardless of this, cities and towns have often been compelled to undergo many hardships because the Courts have generally given private enterprise the advantage wherever the two classes of claims or rights have been in conflict. As soon as a study of preferred uses is undertaken, we must begin to readjust some of our former views. If we say that the first user, as to time when use began, has the first right, should this principle apply to uses of a preferred character? When a preferred use expands and becomes a detriment to a use not preferred, what shall be done in order that the higher use may have protection and the lower use be not damaged or even confiscated? In some States, the preferred users are always protected, regardless of the effect thereof on others. This is probably a wise rule when all users understand the situation from the beginning, and when all investors know that the principle will be carried into effect. Probably the better plan is to have an understanding, with users not preferred, to the effect that as the preferred use grows, the inferior users shall relinquish certain rights and be compensated therefor at a fixed rate, regardless of the time the readjustment may take place, or the demands that may be made on the stream by the preferred user. This should all be done under public supervision. Where water is wasted during times when the supply is inadequate for all, the public should interfere. A city should be permitted to expand its use without being compelled to compensate a user who wastes water. Waste, during times of scarcity, should never occur, under any well-regulated system of public control. By waste is meant a loss of water through carelessness, or such loss as may be prevented by a reasonable expenditure of money in repairs or improvements. There are losses which cannot be prevented, and these we cannot well consider.

Where irrigation is essential to agriculture, the use of water therefore should be preferred, to a limited extent at least. Power can be developed in other ways, and power plants may be located where they will not interfere with the use of water for irrigation. The ideal relation of these uses, geographically, would require the power plants to be located up stream from the irrigation works. For instance, in a mountainous country, the greatest fall is obtained near the headwaters of the stream. This is favorable to water-power plants. If a

reservoir can be constructed on a main stream to equalize the flow of water and produce a constant head for the power plant, which then discharges into a second reservoir where the water may be stored until needed for irrigation, no interference will result. It is impossible to maintain a power plant at maximum efficiency on a reservoir where water is stored for irrigation. Irrigation demands the maximum use of water in a short period during the summer, generally speaking. The maximum demand for power often comes during the winter. The head available, at the reservoir which stores water for irrigation purposes, is at a minimum at the close of that season. The supply of water, therefore, would be irregular for the power plant, and the head would not be uniform. Some of the most disastrous litigation in foreign countries, where irrigation is a necessity, has been brought about by the establishment of power plants which retard or prevent agricultural development.

That preferred uses should be recognized and protected, all must admit. When all facts relating to uses on a single stream are brought together, the administrative officer can determine the relative rights based on other considerations; after which, giving these higher uses appropriate weight, the final adjustment can be made. It would seem difficult to balance these rights, keeping two principles in mind at the same time, but in practice this is not as troublesome as it is in theory. Preferred uses are always comparatively small, and the public generally consents whenever a public authority makes a decision which protects them.

e.—Beneficial use, and the extent thereof, at the time the determination of rights takes place, shall be the measure, the limit, and the basis of the right. Where rights and claims are in an inceptive period, the public record relating to the determination may properly mention them, and all prospective users be given a reasonable time to complete the application of water, under the plans submitted to the office of public record. These incomplete rights should be determined later, as the water is applied to beneficial purposes.

The principle of beneficial use is an important one. When it is applied, every user is restricted to a specific volume, as a maximum, and he can only use so much thereof as can be applied beneficially. Under the Court decisions of the country, generally, claimants are given specific allotments of water, and in many places they are per-

mitted to sell what they find they do not need. This practice breaks down nearly every important principle relating to stream administration. The resulting condition is almost impossible, from the standpoint of the water user. Under the principle of beneficial use, a power plant, for example, is not entitled to the uninterrupted flow of the stream, regardless of the needs of the plant. Under the doctrine of riparian rights, as it is interpreted frequently, a water-power plant can require the entire flow of a stream to reach the plant, regardless of how much thereof can be used beneficially. This permits the plant to waste as much as it pleases, and to bring damage suits, even when it is not injured by users above. If the right of the power plant is limited by the volume it can use beneficially, the owners thereof have no control of or interest in the volume that the stream furnishes in excess of the capacity of such plant.

f.—A declaration of abandonment of rights to use water must be made where water is not used for a certain specified period. The application of this principle makes it impossible for those having water rights simply to hold them without using the water. This rule is incorporated in the statutes of many Western States, yet its practical application is always hampered, because the administrative officers do not generally exercise control, and it is indeed seldom that public, rather than private, interests are upheld. We have a network of theories relating to abandonments, and though these are interesting to study within doors, they are worth but little in the field. The Courts have held almost unanimously that "to constitute abandonment there must be a concurrence of act and intent, the relinquishment of possession, and the intent not to resume it for beneficial use, so that abandonment is always voluntary, and a question of fact".* This means that a user, who has been accorded a right to apply water to some beneficial purpose, may cease all use for as long a period as he may desire, and the actual abandonment will only take place when he admits the same. This is a poor rule. When the use ceases for a limited period, this should be taken as evidence that the water right is of but little value to the former user, and he should be deprived thereof. The public is concerned in use, not in speculation. A definite period, quite brief, should be fixed by law for the interval of non-use. Where the use of

* Wiel, "Water Rights in the Western States."

water is suspended, for such a time, as a matter of convenience rather than of necessity, the right should be canceled, and those who can apply the same beneficially should be favored. The application of this principle emphasizes the principle of public control, so that it has a secondary as well as a primary value.

There are other minor principles that might be discussed. The more important general principles have been already referred to, and a law, State or National, which embodies them, will result in an equitable administration of streams. These principles are not difficult to understand. Injustice is always done when they are obscured by statutes which relate only to procedure, and Court decisions that deal with fine-haired theories. It may be said that, had these wise principles been first incorporated in the law, the Courts might have been able to "adjudicate" the various rights in a satisfactory way. The West has tested this procedure thoroughly. All such trials have resulted in failure. Reference will be made to a single Court decision, made after the doctrine of riparian rights had been abrogated. Where irrigation is practised, a large number of claimants are generally represented. Table 1 is given, without referring to any particular State or stream, yet the data are taken from the Court records. Instead of giving the names of the water users, or the ditches, the writer will simply list the adjudicated rights in order of "priority" numbers.

The adjudication was made by a judge who took pride in his ability to frame water-right decisions. If this decree was based on any principle, the latter has been carefully concealed. An examination of the table will show that there is no relation between the area irrigated and the quantity of water allotted. It would seem that the Court permitted all kinds of testimony to be given. That a large part of this was erroneous, has been demonstrated by later investigations. Surveys have shown that the areas claimed to have been irrigated are inaccurate in practically every case. In an action of this kind in the Court, the facts are often gradually concealed by a record which contains quibbles of attorneys, statements of witnesses that are valueless, rulings and motions that are to no purpose, and testimony which has no bearing on the questions to be determined. Often the record becomes so voluminous and obscure that the Court is afraid of it, and the case is indefinitely delayed.

TABLE 1.

Priority number.	Acres irrigated.	Allotment of water, in cubic feet per second.	Area, per cubic foot per second (Column 2 ÷ Column 3).
1	160	8.65	18.4
2	1 200	67.03	17.9
3	800	30.00	26.7
4	150	12.20	12.5
5	800	37.40	21.4
6	400	22.32	17.9
7	3 300	33.55	99.0
8	300	28.00	10.7
9	300	8.40	35.7
10	200	17.90	11.2
11	300	2.35	127.0
12	300	2.25	133.0
13	160	3.6	44.5
14	.....	32.0	.....
15	600	12.32	48.7
16	160	3.60	44.3
17	1 500	26.50	56.6
18	1 160	84.45	13.7
19	21 000	210.00	100.0
20	800	25.00	32.0
21	2 000	60.10	32.2
22	3 000	78.11	38.4
23	100	10.50	9.5
24	160	17.50	9.2
25	800	12.00	66.8
26	4 000	45.72	89.1
27	1 200	25.00	48.0
28	.....	13.00	.....
29	100	10.00	10.0

In another case a State made an attempt to compromise between public control of streams, through an engineering administration, and Court supervision. Under this plan, the engineering administration is to obtain the facts and the Court is to make all decisions. This compromise was not accepted by the engineers leading in the movement without protest, but it was the best that could be obtained. This law has been tried out. The facts were determined on a single stream and all its tributaries. Complete tabulations and other needed information were submitted to the Court. This basic testimony has been in the hands of the Court for some 7 years, and nothing has been done. It is not too late, fortunately, for the public to provide administrative officers to take charge of this important work. The change can take place without a revolution, except in methods. Many countries and some of our most progressive States have already taken this step. The Courts have not been offended. On the other hand, prominent jurists have assisted in the organization of such administrative systems.

Within the limits of a municipality, where public rights and public interests are always more or less apparent, we unconsciously acquiesce

in the application of many of the principles discussed in this paper. We admit that the city owns and operates the water system. This is public control of the water supply within the city limits. An individual water user cannot sell water to his neighbor. The individual cannot say that his right to use is limited only by the volume of water that may be discharged on his premises or adjoining property. Waste is not permitted where it can be controlled. The parallel might be carried farther. We find some of the principles recognized in national and international laws relating to navigation.

Laws should be framed for the purpose of carrying principles into effect. Health laws generally define principles. When an epidemic threatens a community, the laws give the health officers authority to act at once. They may interfere with the liberty and the pursuit of happiness of many people, yet we hear but little complaint. Principles are being applied, and their value is generally recognized. As a rule, experts in science, rather than in law, frame these statutes and, in a large measure, interpret them. They know what principles should govern, and the public is willing to permit these specialists to lead, provided the important, fundamental principles be upheld. In this work, the law is incidental. The principles embraced therein are of prime importance, and we seldom question any reasonable interpretation of the law. We are thankful that public health can be protected without the necessity of preliminary litigation. We are thankful that these officers can act without great danger of an injustice being served at a critical time.

Engineers, who are placed in responsible charge of streams, belong to the same category. In fact, they must co-operate with the health service and with other administrative officers. The law, to them, consists of public directions as to their duties and, above all, plain definitions of the governing principles.

The administration of streams lies essentially within the field of the engineer. The engineer understands and appreciates the physical problems that are encountered along every watercourse. He must measure the discharge of the streams and the capacity of the diversion works. He must determine the value of water-power plants and ascertain the area and value of irrigation lands. He must estimate the volumes of water that municipalities require and appreciate the relation that should exist among these various uses.

Having all field data before him, and understanding the history of governments which have solved the problems presented, he should be able to frame laws defining the principles which should apply. These laws should be brief, and the principles should be defined in such terms as to leave no doubt as to the meaning of the phrases used. The principles should be stated first, because there is no excuse for an administration until these have been selected and defined. The officers provided in the statute then have a purpose. The foundation for an organization and for just compensation has been laid. No relief has come in any country until those understanding the physical problems have led in a campaign for laws embracing essential principles. The history of the Old World is being repeated in the New. The engineer receives but little financial reward for service of this kind. It is possible that he may secure more remunerative employment under a system which gives no protection to legitimate water users. The Profession, as a body, has never avoided a plain duty because of mercenary motives. Our great engineers are loved and remembered for the service they have performed, not because of the compensation they have received.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE CONSTRUCTION OF THE KLONDIKE PIPE LINE.

By W. W. Edwards, Assoc. M. Am. Soc. C. E.

To be Presented September 2d, 1914.

Synopsis.

Recent discussion of Alaska and the determination of the Government to undertake development work there has prompted the writer to submit the following paper, as describing conditions under which one job of heavy construction in the Far North was carried out. Detailed costs, he regrets to say, were not available, but wages and conditions are described and, whenever possible, day's progress figures are given.

A detailed description of methods used in construction is given, which may serve as a guide or as a comparison for other engineers engaged in laying heavy steel pipe.

The paper describes:

(1) Conditions of climate and geography peculiar to the Far North and affecting engineering work there. In particular, the perpetually frozen ground is mentioned.

(2) Details of construction.

(3) The difficulty of getting satisfactory foundations.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

(4) Points disclosed in the test and operations of the pipe line.

The Klondike pipe line was built during 1907 and 1908 by the Yukon Gold Company, at Dawson, Yukon Territory. It is an inverted siphon crossing the Klondike River Valley, and was designed for a capacity of 125 sec-ft., which figure, however, has been slightly exceeded in operation. By reference to the profile, Fig. 1, it can be seen that the maximum head on the pipe line is about 1 100 ft. The total length of 15 760 ft. is made up of: 2 390 ft. of wood stave pipe, 49 in. inside diameter; 5 850 ft. of riveted steel pipe, 49 in. inside diameter; and 7 520 ft. of lap-welded steel pipe, 43 in. inside diameter. The steel pipe ranged in thickness from $\frac{3}{16}$ to $\frac{11}{16}$ in., and came in lengths of from 15 to 30 ft. It is the largest of several inverted siphons along the line of the Main Ditch System of the Yukon Gold Company, 70 miles in length, which supplies water for hydraulic mining operations on Bonanza Creek, near Dawson.

The location of the pipe line was determined chiefly by two conditions: It was neces-

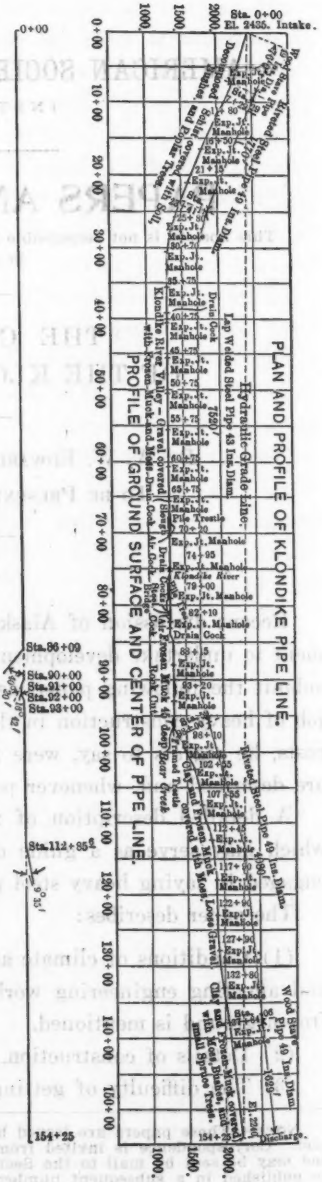


Fig. 1.

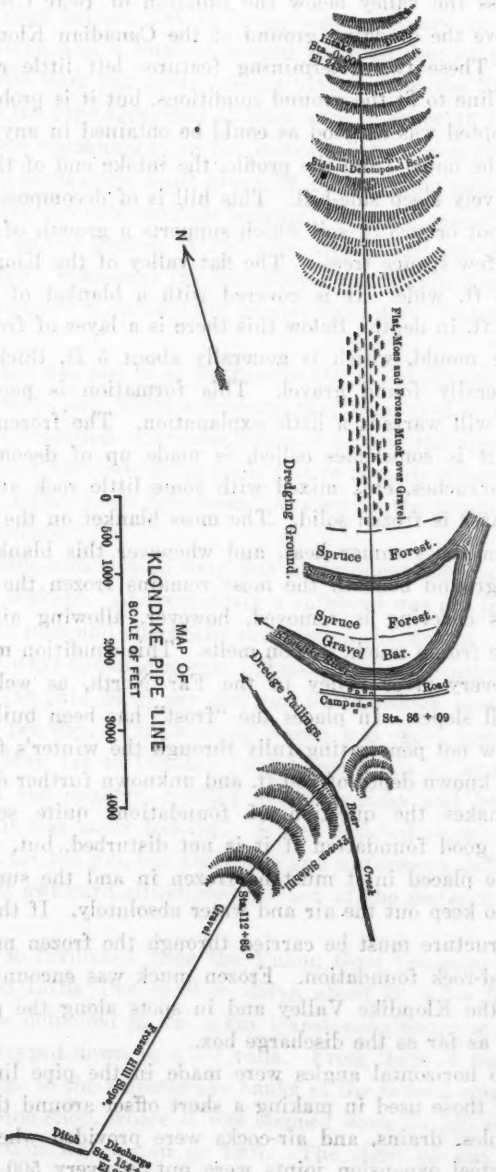

N
Flat-Moss and Frozen Muck over Gravel.
Dredging Ground.
Spruce Forest.
Spruce Forest.
Gravel Bar.
Road
Dredge Tailings.
Sta. 86 + 09
Pier
Creek
Sta. 112 + 86.6
Ditch
Discharge
Sta. 154 + 25
El. 2240.
0
500
1000
2000
3000
4000
SCALE OF FEET
MAP OF
KLONDIKE PIPE LINE

FIG. 2.

sary to cross the valley below the junction of Bear Creek, and also to keep above the dredging ground of the Canadian Klondike Mining Company. These two determining features left little room for adjusting the line to fit the ground conditions, but it is probable that the location adopted was as good as could be obtained in any case.

As may be noted from the profile, the intake end of the pipe is on a comparatively steep side-hill. This hill is of decomposed schist covered by a foot or two of soil which supports a growth of bushes, poplars, and a few spruce trees. The flat valley of the Klondike is here about 5 000 ft. wide. It is covered with a blanket of Arctic moss from 1 to 2 ft. in depth. Below this there is a layer of frozen "muck", or vegetable mould, which is generally about 5 ft. thick, and below this is generally found gravel. This formation is peculiar to the North, and will warrant a little explanation. The frozen "muck", or "frost", as it is sometimes called, is made up of decomposed roots, moss, tree branches, etc., mixed with some little rock and sand, and the whole mass is frozen solid. The moss blanket on the surface protects it from the summer heat, and whenever this blanket is undisturbed the ground beneath the moss remains frozen the year round. If the moss covering is removed, however, allowing air and water to act on the frozen muck, it soon melts. This condition may be found in almost every river valley in the Far North, as well as on the sheltered hill slopes. In places the "frost" has been built up by the seasonal thaw not penetrating fully through the winter's freezing, and extends to a known depth of 200 ft. and unknown further depths. This condition makes the question of foundations quite serious. The "frost" is a good foundation if it is not disturbed, but, if disturbed, the structure placed in it must be frozen in and the surface mossed over again to keep out the air and water absolutely. If this cannot be done, the structure must be carried through the frozen muck to good gravel or bed-rock foundation. Frozen muck was encountered all the way across the Klondike Valley and in spots along the pipe line up the side-hill as far as the discharge box.

Only two horizontal angles were made in the pipe line, with the exception of those used in making a short offset around the base of a hill. Manholes, drains, and air-cocks were provided wherever necessary. Cast-steel expansion joints were put in every 500 ft. to allow for the expansion and contraction due to the extreme variation in tem-

perature, from 80° Fahr. in summer to —70° Fahr. in winter. One three-span steel bridge, 295 ft. long, was necessary in crossing the Klondike River, and a total of several hundred feet of pile trestle or framed trestle were put in at various places along the line. The aim was to have the pipe supported at least once in every 16 ft. Whenever the conditions warranted it, as for instance, when good foundation was deep, trestle bents were built every 32 ft. and king-post trusses were put in to give the intermediate bearing. In good material, sills were merely laid in the bottom of the pipe trench and on these the pipe was supported. Whenever the foundation was within 3 or 4 ft. of the pipe, crib supports were used; if greater than 4 ft., framed bents were used.

One anchor was put in at least every 500 ft., about midway between expansion joints, and, where necessary, they were used more frequently. Each anchor consisted of a cable wrapped around the pipe in the form of a clove hitch, the ends being made fast to two "deadmen" on opposite sides of the pipe. Each "deadman" was set in the ground about 6 ft. below the surface.

On the side-hills drains were dug at frequent intervals to lead off seepage and leakage. At every drain a bulkhead was put in which fitted closely to the pipe and effectually intercepted the water.

In many ways conditions were unique for heavy construction. Common labor was paid $4.00 per day and board, which meant a total of about $5.50 per day. The working season was short, being limited to 5 months—May 1st to October 1st. Some kinds of equipment were plentiful in the country, notably steam engines and boilers, but in other lines it was sadly lacking. A well selected stock of equipment was ordered for the job, but breakages and unforeseen difficulties sometimes caused awkward situations, in view of the distance from a base of supplies.

Power was furnished from the Yukon Gold Company's hydro-electric plant on Little Twelve Mile River, about 50 miles from the pipe line. It was delivered at the main transformer-house at 33 000 volts, and there stepped down to 2 300 volts. From there it was distributed at this voltage to the transformer banks at the several compressors and at the machine shop, where it was stepped down to 220 volts.

Construction was begun in 1907. The right of way was cleared and a small quantity of pipe trench was dug. The three-span steel

bridge over the Klondike was erected. Foundation cribs were put in the river bed, the work being done mostly while the river was frozen over and the water low. Piles were driven in the river bottom and sawed off at the gravel surface, and on them the cribs were built. These cribs were then extended to a height of about 5 ft. above high-water level and filled with rock. They were sheathed with sheet iron on the up-stream end to protect them from ice. Derricks were used in assembling the steel spans. During this season, also, a pile approach to the bridge, 400 ft. long, was built, and several other smaller pile trestles and framed trestles. Construction was halted in the midst of the working season in 1907.

In the spring of 1908 construction was begun again. It was desired to have the job completed by the following October 1st, the end of the working season. All the pipe was not then on the ground, and what was on hand was not continuous. That is, sections of pipe were missing here and there. As there were so many different kinds of pipe in the line, and as extra pieces were few, it was generally impossible to use sections of pipe interchangeably, therefore work could be started only at certain places. In the course of the work, it was necessary to start laying pipe at four different stations. An error in surveying would lead to awkward consequences where any of these headings joined, as well as at angle points in the pipe line. As there was no heavy machinery or plate for making pipe of this size in the country, and there would not be time to send "outside" for extras, it was necessary to be very careful in surveying.

All the pipe on hand was first measured up carefully and a relation established between actual measurements and those given on the blue prints. An offset line was then run paralleling the located line at a distance of 20 ft. As the seasonal thaw had loosened the original hubs so much that they could not be relied on, measurements were begun from a point on one of the bridge piers. Hubs were driven about 99 ft. apart, and measurements were taken from plumb-bob strings held from tripods. This was done on the side-hills as well as the flats. A standard pull of 15 lb. was used, the same as had been used in measuring up the pipe on hand. Corrections were made for temperature variations. Levels were run over the hubs, and horizontal distances were computed from the slope measurements and elevation differences. This system resulted in giving a first-class base line, which could be

FIG. 3.—VIEW OF INTAKE SIDE-HILL, LOOKING TOWARD DISCHARGE.

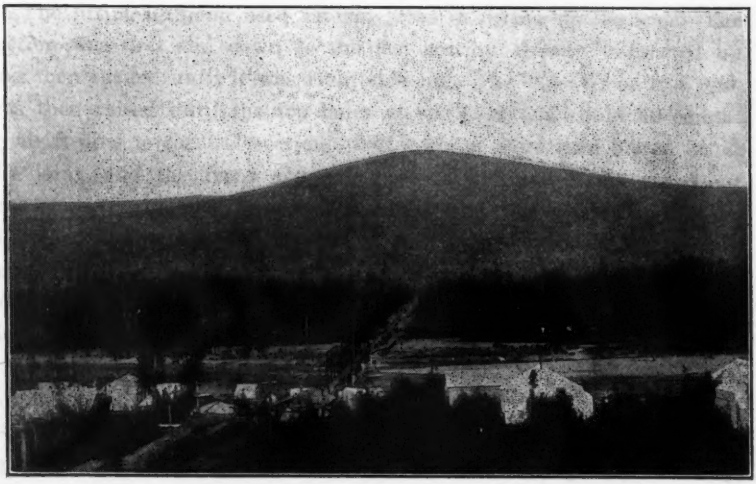
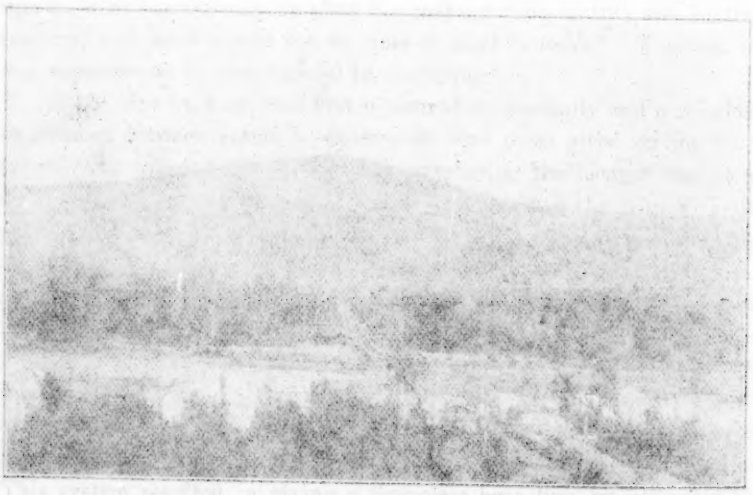


FIG. 4.—KLONDIKE RIVER AND BRIDGE, LOOKING TOWARD INTAKE.



absolutely relied on in setting grades or starting points. Grades were "shot in" by transit. Points were set along the edge of the pipe trench, and were transferred to the bottom, when needed, by using a large wooden square and a carpenter's level. Bench-marks were set every 50 ft. in elevation. Levels were determined by running ahead for half a day and then tying back to the starting point.

In construction, it was frequently found advantageous to depart from the profile in order to avoid excessive excavation, and in these cases grade changes were made by shifting angle points.

Pipe was distributed along the line by a tramway. The cars were pulled by horses across the flat and by hoists on the hills. Of these hoists, two were run by steam, and one by electricity. Signals were given by use of a wire attached to a spring pole. A telephone system also helped in signaling and general communication. Turn-tables were installed on which to change the sections of pipe end for end when necessary. Steel-frame cars were used. In hoisting, a bridle was attached to the end of the pipe, and the cable was shackled to this bridle. When the pipe was at the spot desired, the cars were chained to the rail, and the pipe, with the hoisting cable still attached, was rolled into the ditch. This held the pipe from getting away on the side-hill, and the cable always had so much spring that no harm was done to the engine.

The fitting-up gang used ratchet jacks in fitting up the pipe. The section was first slid down to the last section already connected up and then turned until it was right side up. The "front", or free end, was then raised until the top holes at the back end could be caught by drift-pins to the last section. Guide-straps were then bolted on at the joint, and the front end was lowered gradually. As the front end came down, the joint was sledged and the guide-straps were tightened as necessary. When the joint was entered all around, a ratchet turnbuckle was attached and used to pull it together. Fitting-up bolts were put in every third or fourth rivet hole. Four or five sections of pipe per 10-hour shift was an average for the fitting-up gang.

Riveting, caulking, and drilling were done with pneumatic tools. Air was furnished by 8 by 8-in. Clayton air compressors run by 15-h. p., 3-phase motors. One air compressor would furnish air enough to operate a riveting hammer, caulking hammer, drills, and air dolly, with not much to spare. The compressor outfits were mounted on

platforms and moved from place to place as desired. The riveting of the holes in the bottom of the pipe was done from the inside, and of the upper holes from the outside. Air dollies were used in holding on inside the pipe and spring dollies on the outside. Each riveting gang consisted of a riveter, bucker-up, heater, and passer. An average day's work was from 160 to 180 1-in. rivets, when conditions were good. Fitting-up bolts were used liberally, and the joint was well sledged before driving each rivet. All castings were hand-riveted to the pipe, the riveting being done on the inside.

The expansion joint castings were frequently found to be too large for the pipe. In such cases the pipe was heated by a blast lamp and belled out with hammers to fit the casting. Rivet holes in all castings were drilled in the field after assembling. All caulking was done with rounded-edge fullers, the square tool being dispensed with. The riveted pipe was caulked on the outside only, the lap-welded pipe was caulked both inside and outside. Rivets were inspected with a 1-lb. tapping hammer.

The pipe was coated on the outside with a red mineral linseed oil paint and inside with a quick-drying black paraffin paint.

The wood stave pipe was of California redwood staves thoroughly seasoned. It was laid in a trench 2 ft. deep, and, when completed, was back-filled to a depth of half the diameter of the pipe, so as to give a good bearing.

The intake penstock was equipped with overflow and turn-out, as was also the discharge box. The discharge box was located on a shaded side-hill on frozen clay. This condition made necessary considerable cribbing to protect it from the sloughing of the clay when it thawed consequent on the removal of its moss covering.

Timber was secured from adjoining leases and also from river drives. It was necessary to do considerable work to protect the river banks from undercutting in the vicinity of the bridge, as they were of loose gravel, loam, and moss. Piles were driven and shear cribs put in along the river bank above the bridge. Protection cribs were also put in above the pile bents on the bridge to take the brunt of the ice pressure and to break up the ice stream.

Frozen muck, reaching to a depth of 45 ft., was encountered between Stations 84 and 90. Shafts were dug to gravel foundation and trestle bents were put in at intervals of 32 ft. Between Stations 90 and 93

a heavy cut through dredge tailings and solid rock was necessary. This was just at the base of a hill which was covered with frozen muck. The digging of this section was delayed until as late as possible so that the extreme cold would keep the overburden on the slope above from thawing on exposure, and consequent sliding.

A small spot of frozen clay was encountered at about Station 107, and the depth of good foundation being unknown, a corduroy of spruce trees was laid on the frozen clay in the bottom of the pipe trench and this was covered with about 2 ft. of gravel. A bulkhead was built around the pipe above this place to shut off all seepage and leakage, and a drain was run off to the side. The pipe was back-filled and the surface covered with moss again. Apparently, the foundation remained frozen, as no settlement was noticed.

Although work was begun at the camp about April 1st, it was not until about May 15th that all the snow was off the ground and it was possible to work to the best advantage. A night shift was put on about May 15th and continued until October 1st, by which date the pipe was all laid. The long days of the extreme northern latitude made lights unnecessary on the work until August 1st. After that, a system of arc and incandescent lights was installed, which worked very well. The missing sections of pipe began to arrive about July 1st, and from that time onward operations were rushed. A force of 350 men was used to push the work to completion. The different headings of pipe were connected up at the expansion joints. This was effected by sliding the outside ring of the expansion joint back on the pipe and then pulling it up to position after the pipe had been lined up.

A tabulation was made showing the space to be left open at the expansion joints at various temperatures, so that they would not close during the hottest or open in the coldest weather. This was followed in setting the joints. Gauges were wired on at each expansion joint so that unequal expansion or contraction could be noted and steps taken to prevent it. The coupling bolts for each expansion joint were set so that the lap in each joint could not be less than 1 in. Reference points were also put in on each 500-ft. section of pipe, so that movement could be detected. Anchors were put in at every angle point. These were generally either bulkheads of timber against the pipe, used as thrust anchors, or cables around the pipe attached to "deadmen." When possible, at the angle points, a timber crib was put completely

over the pipe and filled with rock. The high cost of transportation and the character of the foundation made concrete anchors out of the question. The only timber available was white spruce, which was soft and brittle, and had to be used with a high factor of safety in all structures.

Some trouble was experienced from the lap-welded pipe. Two makes were used on the job. One, an American make, was perfectly satisfactory, but the other, a German make, was not. One section of the second kind cracked along the weld for a distance of 10 ft. on being dropped into the pipe trench. This was patched and reinforced, and gave no trouble in operation. Another section of this pipe burst in testing under 835 ft. head. Examination showed a flaw in the metal. When it burst, the water was shot out to the side of the pipe line and little damage was done. This was the extent of the trouble with the line in testing.

In operation, the quantity of water flowing through the pipe fluctuated considerably. As the difference in elevation between intake and discharge was 195 ft., this variation caused great fluctuations in the water level in the pipe at the intake end, there being no regulating gate at the discharge end. When little water was flowing, it ran in a very rapid stream down the bottom of the wood stave pipe at the intake end. This had the result of rapidly wearing out the bottom staves, so that considerable patching had to be done.

Bulkheads were put in at each end of the pipe and all other openings were closed as soon as the pipe was thoroughly drained. Thus the air circulation inside the pipe was shut off and the subsequent snow formed a blanket which protected it from the extremes of the winter temperature. It was found that with an air temperature of — 30 or — 40° Fahr. the temperature inside the pipe was only about — 5° Fahr. In operation, the pipe is drained at the close of the working season, about October 1st, and opened again about May 1st.

The peculiar and therefore the most interesting feature of this work was, as noted before, the study of foundations and the best styles of substructures to be used in the frozen ground. In places in the North, piles of saw-dust have been placed on the moss, and buildings put on this foundation. When water is kept away, this makes a very good foundation, as the saw-dust protects the moss and the frozen ground below from heat and consequent thawing, thus insuring

stability. On this same principle, in running a road or railroad grade across frozen ground, it is best not to cut but rather to make fills on top of the moss. Thus, on a railroad grade near Nome, a fill was made of peat cut from the surface of the tundra. When dried out and packed this gave no more trouble—until it burned up—whereas a cut in frozen ground is but an invitation to perpetual trouble from slides and settlement. In flume substructures, on the Yukon Gold Company's line, on frozen side-hills, cripple bents were tried and discarded in favor of level mudsills set in the "frost", about 2 or 3 ft. below the surface at the down-hill end. These were mossed over and remained frozen unless excessive leakage thawed the ground.

Mr. A. C. Strong, of the Yukon Gold Company, developed a type of structure which suited the foundation very well. He used piles driven into holes previously thawed out by steam points. The steam point is a strong pipe, at one end of which steam is introduced. The opposite end is rounded and a small hole left through which the steam may escape. This point is started into the frozen ground and is pounded down with a hammer as fast as it thaws through the "frost" in its way. These steam points were developed and used to considerable advantage in mining the frozen gravel. In substructure work, the steam point is used until the "frost" is thawed to the desired depth. It is then removed and a pile driven into the thawed-out hole. As the surface moss is disturbed hardly at all in this process, the pile immediately freezes in and remains in this condition. At last reports, pile substructures, such as these, were giving good service. This appears to be the cheapest form of substructure as yet devised for use in ground frozen to any considerable depth.

The writer has seen but little in technical literature dealing with construction in the Far North, and hopes that some discussion may be stimulated by this paper, especially on the subject of structures founded on frozen ground.

The foregoing work was done by day labor by the Yukon Gold Company, Mr. O. B. Perry, General Manager; C. A. Thomas, M. Am. Soc. C. E., Resident Manager; H. H. Hall, Assoc. M. Am. Soc. C. E., Superintendent of Ditch Construction, and in general charge of surveys; Mr. C. G. Newton, Superintendent of Pipe Lines; and the writer, Division Engineer and General Foreman.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE CONSTANT-ANGLE ARCH DAM.

By Lars R. Jorgensen, Assoc. M. Am. Soc. C. E.

To be Presented September 16th, 1914.

Synopsis.

There are two features which distinguish this type of dam from the ordinary arch dam. These are (1) "Economy of Material", and (2) "Ability of the Dam to act as an Arch, even Close to the Foundation, to a Much Greater Extent than the Ordinary Arch Dam." It is understood that the canyon is wider at the top than at the bottom, which is generally the case.

The first claim, "Economy of Material", is proved in the following manner. It is shown that any arch slice closing the gap between two abutments must subtend an angle of (133° theoretically) about 120° for greatest economy of material.

The constant-angle arch is designed to comply with this law as closely as possible at any elevation from crest to foundation. The length of the up-stream radius is decreased toward the foundation in the same proportion as the canyon becomes narrower, thereby keeping the angle subtended by the arch at the most economical value. In the case of an ordinary arch dam built in a V-shaped canyon, the angle subtended by the arch is rapidly getting smaller toward the lower elevations, and therefore the building material is placed in a less

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

economical position and the arch soon becomes so flat that it cannot act as an arch, but the load is carried by shear and cantilever action almost exclusively.

To prove the second claim, attention is called to the fact that the deflection of an arch when loaded is proportional to the square of the length of the up-stream radius. This radius may be several times shorter at the foundation than at the crest of the dam in the constant-angle arch design. If the radius should happen to be, say, four times shorter at the foundation than at the crest, the deflection required at the foundation would be sixteen times less than at the crest, using the same unit stresses. In such a case, the arch would be able to support sixteen times as much load at the bottom as an ordinary arch having a constant up-stream radius.

In order to obtain equal safety at all points, it is necessary that the thickness of the arch should be made the greatest in the middle, and diminish toward the abutments, especially toward the lower elevations. This is on account of the fact that an arch dam when loaded acts like a long column in compression, and such a column is weakest in the middle; besides, the central portion of the arch moves back and forth according to different load conditions, and for that reason alone the unit stresses used for this portion should be kept lower than for the more stationary portions of the dam.

It is also pointed out in the paper that the initial stresses (Poisson's ratio), due to the water pressure and to the weight of building material, can be utilized to carry a portion of the load, and that these stresses will carry the greatest proportion of the load in arch dams designed according to the constant-angle arch principle.

An approximate short-cut method for finding the proportion of the load taken by the arch and by the cantilever, respectively, is given, also some information as to the maximum foundation pressure at the toe, and the shear existing at the foundation. Finally, a few practical examples are described.

It has been the object of the writer to bring out a dam design especially adapted to high and comparatively narrow canyons. For such a condition, the constant-angle arch dam will generally show a saving of material of 33% or more over an ordinary gravity dam, and at the same time it will possess a factor of safety more than twice as great as that of the gravity dam.

This method of arch dam design shows how it is possible to make the dam take most of the load, acting as an arch, leaving only a small portion for cantilever or gravity action.

There is no single type of dam which can be readily adapted to every site, therefore a number of types have naturally been developed and applied to satisfy the different demands as these have presented themselves. This statement applies to the type of arch dam to be described in this paper, as well as to any other.

All masonry dams must be built on good rock foundation, with the exception perhaps of those of the Ambursen type, the requirements for which are not so exacting. Where the dam site is narrow and more or less V-shaped, a single arch dam of the type to be described can be built to advantage. How narrow the site must be cannot be stated in general terms, for a few calculations will have to be made to show whether there is economy in using the arch dam rather than any other type. Many dams having gravity sections have been built curved in plan, this curved form giving them considerable extra strength at the crest, especially when the arch subtends a large angle. However, the ordinary arch form gives but little extra strength at the bottom of a V-shaped canyon, because, no matter how large an angle is subtended by the arch at the crest, this angle has become small at the bottom. In other words, the arch cannot carry much load on account of its flatness. This non-action of the arch at the bottom the following method of design seeks to eliminate, besides giving a most economical distribution of material. It does not matter whether a gravity section or a much thinner one is used, the arch can be forced by the design to take up the greater portion of the load. That this is the most economical way of loading the structure should be readily admitted, as arch stress is distributed nearly evenly over the whole section, whereas the stress due to gravity action is very poorly distributed, being a maximum along the down-stream face, and diminishing to approximately zero at the up-stream face. These two stresses are in two different planes, 90° apart, thereby tending to give lateral support to each other, as both act at the same time.* This indicates

*This conclusion can be fairly well proved by the fact that when testing a cube or column of any material in compression, it swells laterally in a plane 90° from the plane of the compressive force. With lead, this is quite apparent. If this swelling be prevented, the crushing strength of the material will be materially increased.

that it is not economy to eliminate entirely the cantilever or gravity action in an arch dam, even if this were possible, as it acts like the hooping in a column, giving lateral support to the dam body. The arch action, however, must predominate, on account of the more economical distribution of stress over the section, and also in order to eliminate the large shear action otherwise introduced by the force causing the bending of the cantilever.

When an arch dam is loaded, the length of the arc is bound to become shorter, due to the deformation of the material caused by the axial stress. The only way the structure can compensate for this shortening is by deflecting down stream. If the canyon has a fairly regular shape, this deflection will be greatest at the crown of the arch and will decrease to zero at or near the abutments. This deflection of the crown introduces cantilever stresses which ordinarily reach their maximum values in the middle section, where the dam is highest and the deflection is greatest. How the load divides itself between the arch and the cantilever depends on their relative ability to carry load, and it is easily seen that if we wish to design an arch dam where the arch is to carry the greater portion of the load, we must keep the arch deflection down to a minimum, and especially at and toward the bottom, where the dam is fixed in position and cannot move. Then, too, the arch should be designed so that the amount of this deflection decreases at a uniform rate from a maximum near the crest to a minimum, as close to zero as possible, at the foundation, in which case any imaginary horizontal arch slice deflects as if it were actually independent and free to move relatively to any other slice above or below, thereby eliminating undesirable internal stresses.

In order to obtain a preliminary dam section for any given dam site the simple formula,

$$t = \frac{P R_u}{q} \qquad (1)$$

can be used for finding the thickness of a sufficient number of arch slices at different elevations. In this formula, t equals the thickness of the dam, in feet, at any given elevation; P equals the water pressure, in pounds per square foot; R_u equals the length of the up-stream radius, in feet; and q equals the average stress, in pounds per square foot, of the area of the dam section (Fig. 1).

It will now be shown how to obtain the most economical arch section, that is, a section where the material is placed so as to take up the load with minimum resulting stress. From the formula, $t = \frac{P R_u}{q}$, it is seen that the thickness, and therefore the area, of the dam section, varies in direct proportion with the radius. The volume of concrete in any arch dam is equal to the area of the section multiplied by the length of the mean arc. The length of the mean arc can be expressed as the length of the mean radius, R_m, multiplied by the subtended angle, in terms of π,

$$\text{or, } V = \text{Area} \times R_m \times 2\theta \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where 2θ is the subtended angle.

The mean radius, R_m, equals half the width, W, of the span divided by the sine of half the subtended angle (Fig. 1). Thus

$$R_m = \frac{\frac{1}{2} W}{\text{sin. } \theta} \ldots\ldots (3)$$

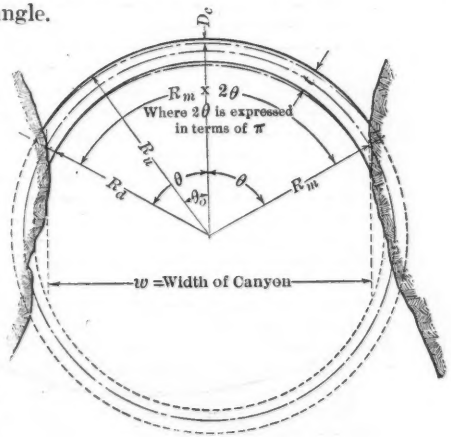


FIG. 1.

As the area of the section is proportional to the length of the radius (both to R_u and R_m), Formula 2, for the volume of masonry can be expressed thus:

$$V = C \times \frac{\left(\frac{1}{2} W\right)^2 \times 2\theta}{\text{sin.}^2 \theta} = \frac{K \times \theta}{\text{sin.}^2 \theta} \ldots\ldots\ldots\ldots (4)$$

where C and K are constants, the latter depending on the width of the canyon.

According to Formula 4, the volume varies with the term, $\frac{\theta}{\text{sin.}^2 \theta}$. The differential coefficient of this term, equated to zero, gives the minimum for a central angle of 133° 34′, which means that any horizontal slice of the dam has the least volume when $2\theta = 133° 34'$. In other words, the dam contains a minimum quantity of material when the central angle is kept at 133° 34′ at all elevations.

The curve on Fig. 2 shows this graphically; the abscissas represent the central angle, 2 θ, and the ordinates represent the term, $\frac{\theta}{\sin.^2 \theta}$, the latter being proportional to the volume of masonry. In addition to showing the point of maximum economy, this curve also shows that, as long as the subtended angle, 2 θ, is kept between the limits, 150° and 110°, the variation in the volume of masonry is very small, but that, outside these limits, the volume increases rapidly. Fig. 3 illustrates how the dimensions of successive arch-shaped slices of a dam are determined and superposed, one upon the other, to form the structure. Six elevations have been established, forming Contours

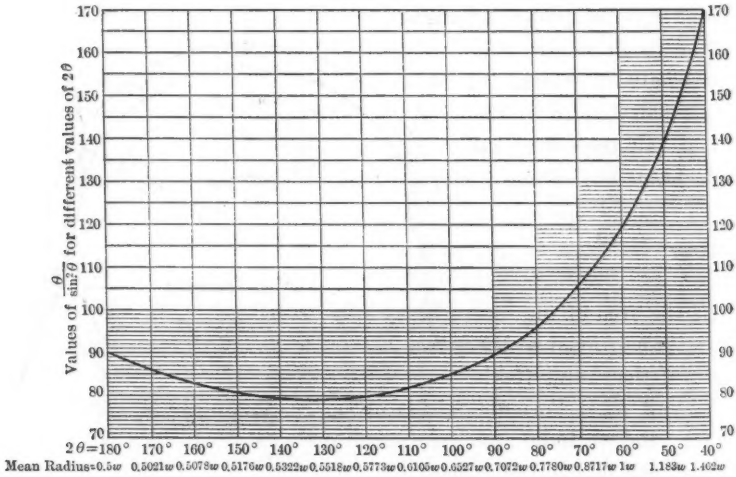


FIG. 2.

I, II, III, IV, V, and VI, Contour VI corresponding to the level of the water retained by the dam, and Contour I corresponding to the lowest water level. The distance across the canyon at the top of the dam, W_6, is measured on a map to scale. Then the mean radius, R_{m6}, which will give the dam the greatest strength with the least volume of material, is found by substituting the most economical central angle, 2 θ, in the formula $R_{m6} = \frac{\frac{1}{2} W_6}{\sin. \theta}$. The thickness, t_6 may now be determined algebraically from Equation 1. However, for the first calculation, the length of the up-stream radius will have

to be estimated from the known length of the mean radius, but this is not difficult.

At Contour V the distance, W_5, across the canyon is measured, and the most economical radius and thickness of section at this elevation is found in the same manner as for Contour VI. The center of curvature of the arch slice at Elevation V does not necessarily lie on the same center line as the center of curvature of the arch slice at Elevation VI, as perfectly symmetrical slopes of the canyon sides will rarely be found. It is desirable, however, for practical construction and appearance sake, to try and keep as many centers

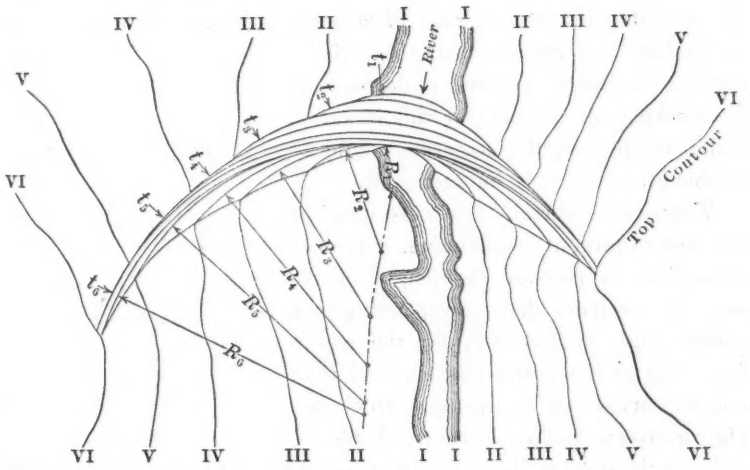


FIG. 3.

as possible on common center lines, as shown on Fig. 3. The radii and thickness at lower elevations are determined in a similar manner. It is thus seen that the main requirement which makes for high economy of material is to keep the central angle of the arch as near to the most economical value at each elevation as the contour of the site will permit. This necessitates the abandonment of the constant up-stream radius generally used in arch dams as at present constructed, and the substitution of radii of varying lengths corresponding to the widths of the canyon at different elevations. In some localities it may be desirable to face the dam with stone, and it may then be of advantage to step the faces. It will then consist of a number of

cylindrical rings superposed one upon the other. The up- and down-stream radii in such cases are ascertained in exactly the same manner as given above.

To prevent upper portions of the dam from overhanging lower portions, it will be necessary to have the thickness of the section increase from the crest toward the foundation. The proportional increase in water pressure, therefore, must be greater than the proportional decrease in length of the up-stream radius toward the foundation. The ratio of increase in water pressure is always fixed, and the ratio of decrease in the length of the up-stream radius depends on the slope of the canyon sides. If these slopes are such that at any intermediate elevation the ratio of decrease in length of the up-stream radius has been greater than the ratio of increase in water pressure, a decrease in the thickness of the dam at this elevation would result, and the structure would be overhanging, which is impracticable.

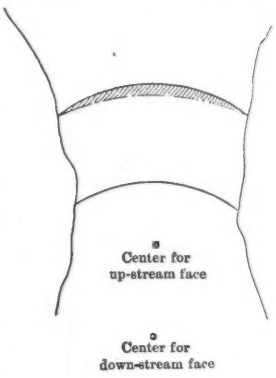


FIG. 4.

Whenever a certain thickness must be provided to prevent overhanging, it is most economical to increase the length of the mean radius above that corresponding to a central angle of 133° 24′, for the reason that a flat arch requires less material than a more curved one of the same thickness. The up-stream radius, however, should be kept small, and not have a center common with the mean radius, and of such length as to allow the deflection line (*A*, Fig. 9) to become one continuous straight line from crest to foundation. The material added is small (as shown by the cross-hatched spherical triangle in Fig. 4), as such additions are generally required only near the foundations, where the arch is short.

In the foregoing, the thickness of the different arch slices has been determined as if all the load were taken by the arch, and the dam section had no gravity action at all. How safe a dam would result depends primarily on the unit compression allowed when using Formula 1 for finding the thickness at different elevations. However, in order to obtain a dam of uniform safety for its whole height, the foregoing design must be somewhat modified to take into considera-

tion the gravity and curved beam action. The foregoing theoretical design would prove to be weakest in the middle in most cases, for the same reason that a long column held at both ends is weakest in the middle, and on account of having the highest cantilever stresses here. Whenever t is small compared with R_u, the arch when loaded is practically a long column in compression, and the length of the arch, therefore, should not be more than twenty-five times its thickness, if the material is to be highly stressed. It is true that this

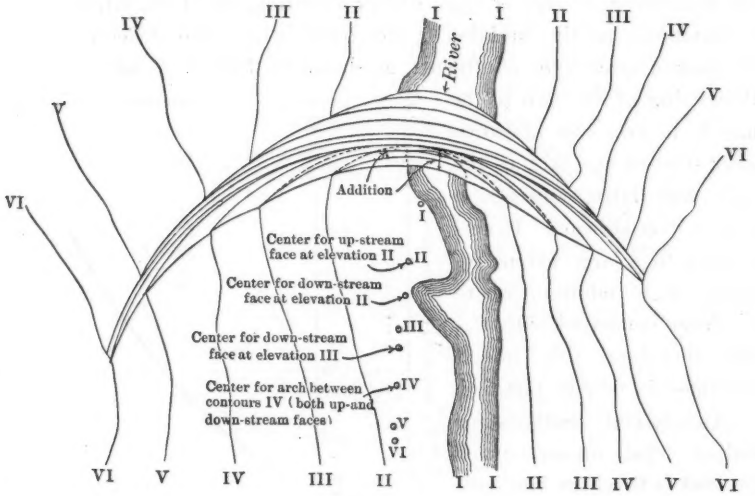


FIG. 5.

circular column is supported to some extent along one side, but this added stiffness may be largely offset by the fact that the water may not soak through the up-stream face uniformly; that is, the effect of the water pressure, in all probability, would be unsymmetrical about the center line of the dam. On a high and comparatively thin arch dam section, the resulting compression due to cantilever action and weight of material above may become excessive near the foundation, requiring some additional material along the down-stream face toward the foundation, as shown by Figs. 5 and 7. On account of the necessity for this additional material, the use of a smaller central angle than the theoretical 133° might actually be more economical, and 120° or even less, in a case like this, might give very satisfactory results (generally greater at the crest and smaller toward the foundation, depending

on local conditions). By decreasing the central angle from 133° to 120°, the volume of material is increased 1% (Fig. 2), but, at the same time, the thickness has been increased 6 per cent. For angles much smaller than 120°, the volume of additional material required increases at a greater rate than the thickness, and therefore a lower limit for the economical value of the central angle is soon reached, and extra material will then have to be added. The thickness of this added material should decrease vertically from a maximum at the foundation to zero at some higher elevation, and horizontally from a maximum in the middle, or the point where the deflection is a maximum, toward the abutments, as shown by Figs. 4, 5, and 7. This thickening of the dam in the middle to take care of cantilever stresses also stiffens the arch materially, considering it as a curved beam. It acts as such to a large extent toward the foundation, where t is large compared with R_u, and therefore this added thickness is doubly justified.

$$CC_c = \frac{\frac{2\sin.\theta}{\theta}\left(1-\cos.\theta\right)+\tfrac{1}{2}\left(\cos.2\theta-1\right)+\left(1-\cos.\theta\right)}{\frac{3\theta}{\sin.\theta}+\cos.\theta-4}$$

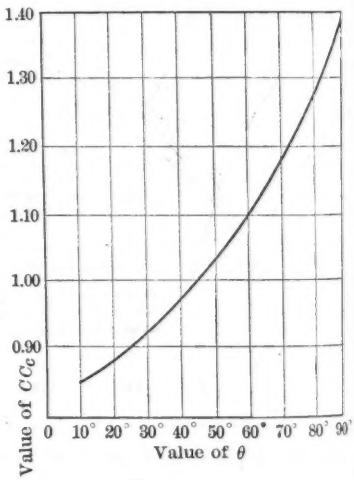


FIG. 6.

Approximate methods for finding what proportion of the load is taken by the arch and by the cantilever have been thoroughly discussed in the paper entitled "Lake Cheesman Dam and Reservoir"* by the late Charles L. Harrison, M. Am. Soc. C. E., and Silas H. Woodard, M. Am. Soc. C. E. The late R. Shirreffs, M. Am. Soc. C. E., in his discussion of that paper develops the following formula for the crown deflection of an arch dam.

$$D_c = \frac{CC_c \times P_1\ (\text{up-stream radius})^2}{E \times t} \qquad (5)$$

where $P_1 = P \times \frac{R_u}{R_m}$, and CC_c is a factor which takes the curved beam action into consideration and can be found directly from Fig. 6, which for the sake of ready reference, and on account of a typographical error in the original formula, is reproduced here from Fig. 22 in

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 89.

Mr. Shirreffs' discussion. E is the modulus of elasticity and t is the thickness. This formula and curve, however, do not take the initial stresses in the dam structure into consideration, and, therefore, before applying this formula for finding arch deflections due to the water load, we must determine how much of this load is carried by the arch due to initial stresses, because, only after deducting this part of the load does the remainder divide up between arch and cantilever action in the ordinary sense.

By initial stresses are meant stresses principally due to the weight of the structure and to the water pressure. Therefore, these stresses reach their maximum values at or near the foundation, and are zero at the crest. Thus far, they have not been much discussed, but they are very important, and should be taken into consideration when attempting to find the actual division of load between the arch and cantilever.

When the dam is completed, the lower layers of the dam body are in compression, due to the weight of the material above. When a body is compressed, the dimension in the direction of the compressive force becomes smaller, but in other directions the body swells, if free to move (lateral strain). The ratio of lateral to longitudinal strain is different for different materials. It is known to lie between $\frac{1}{3}$ and $\frac{1}{4}$ for bodies such as metals, but for concrete, which is not a very homogeneous substance, this ratio is about $\frac{1}{5}$ at the unit compression generally used in arch dam design. A. N. Talbot, M. Am. Soc. C. E., gives* values for this ratio, $\frac{1}{m}$, ranging from 0.1 to 0.18 at low loads, up to 0.25 at loads near the ultimate for 1:2:4 concrete 60 days old. A French commission on reinforced concrete found Poisson's ratio, $\frac{1}{m}$, for ordinary concrete to be about 0.16 when a force of 200 lb. per sq. in. was applied perpendicularly to the largest dimension of the concrete, and about 0.22 when applied at right angles to the largest dimension. Professor C. Bach, of Stuttgart, Germany, found for Poisson's ratio the value $\frac{1}{5.3}$ for 1:2:3 concrete 45 days old at a unit compression of about 180 lb. per sq. in.†

* *Bulletin No. 20*, University of Illinois.

† More details may be found in *Engineering News*, Vol. 68, p. 208.

A dam body often contains considerable quantities of large stones, and the presence of these stones has the effect of increasing the value of Poisson's ratio of the total mass. (Poisson's ratio for rock is about $\frac{1}{4}$.) Therefore the use of $\frac{1}{5}$ for this ratio for the total mass seems to be entirely justifiable. Any horizontal layer of material will have to sustain compression corresponding to the height of the masonry above it, and, therefore, will actually become shorter in a vertical direction and have a tendency to expand horizontally. If the abutments are unyielding, the arch may be prevented from actually becoming longer, in which case axial compression is introduced, the same as if water pressure acted on the structure. The value of this initial axial compression per square unit area is equal to the vertical compression multiplied by $\frac{1}{5}$ if the abutments are unyielding, and, as far as the final result is concerned, it does not make any difference whether they are absolutely unyielding or not.

If the specific gravity of the concrete for the dam is taken at 2.3 and the height of the dam at H, then the average vertical pressure can be expressed as $\frac{2.3\,H}{a}$, where a is the ratio of the total height of the dam to the height of a rectangular wall having the same sectional area and the same base. The ratio, a, is known as soon as the section is known, and in dam design the section must be more or less determined before final calculations can be made.

The dam section shown in Fig. 7 has an area of 9 668 sq. ft., a base width of 70 ft., and a height of 250 ft. The height of the masonry column causing the mean vertical pressure, therefore, is $\frac{9\,668}{70} = 138$ ft., and $a = \frac{250}{138} = 1.81$, making the mean vertical compression on the foundation—in terms of head of water—equal $\frac{2.3\,H}{1.81} = 1.27\,H$, with no water pressure acting on the up-stream side.

The condition of reservoir full introduces an additional force—the radial water pressure—tending to compress the dam body in a direction perpendicular to the direction of the compressive force due to the weight of the body. At the bottom of the dam this force is equal to H, in case the water is standing to the crest of the dam. In this

case the radial water pressure tends to counteract the swelling of the concrete in an up-and-down-stream direction (due to the weight), thereby introducing additional initial axial compression.

The total resulting initial axial compression at the foundation of the section shown in Fig. 7 (in terms of head of water), therefore, is

$$\frac{1}{5}(1.27\,H + H) = 0.454 \times H \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where Poisson's ratio has been taken as $\frac{1}{5}$. Mr. H. Ballet, a French

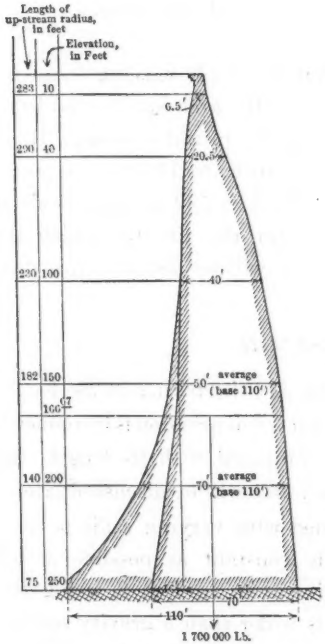


FIG. 7.

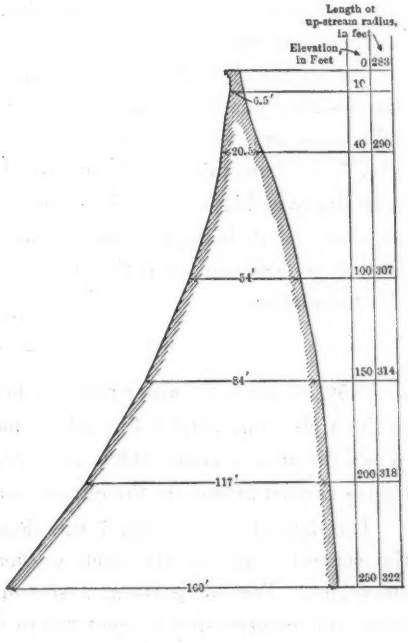


FIG. 8.

engineer, was probably the first to point out the necessity of taking Poisson's ratio into consideration when attempting to find the actual stresses in a dam body.*

The height of water, h, that this initial axial compression of 0.454 $\times$ H will resist without causing any shortening of the length of the arch at the bottom can be found by using Formula 1, thus

$$h = 0.454 \times H \frac{t}{R_u} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII, 1909, p. 51.

For the narrower section shown in Fig. 7, $t = 70$ ft. at the base, $R_u = 75$ ft. Substituting these values, it is seen that this section at the very bottom is able to carry,

$$h = 0.454 \times H \times \frac{70}{75} = 0.425 \times H,$$

or 42.5% of the total head of water, as an arch, before any shortening in the length of the arch occurs.

The initial axial compression holds in equilibrium the stresses due to 42.5% of the total head at the bottom, the remaining 57.5% of the load will divide between cantilever, arch, and curved-beam action, in the usual way.

By analyzing Formula 7 it is seen that by simply varying t and R_u, the designer has it within his power to make the arch and curved beam take more or less of the load by utilizing the initial stresses. If, in Fig. 7, the base thickness is increased from 70 ft. to 110 ft., and correspondingly at higher elevations, and R_u is kept at 75 ft., which is about as short as it is practicable to make it for the site for which the section is designed, then the initial stresses will be able to support at the foundation,

$$0.4 \times H \frac{110}{75} = 0.585 \times H,$$

or 58.5% of the total water pressure before any shortening in the length of the arch occurs, and before additional axial compression is introduced. When the arch becomes thick, however, compared with its length, the load is carried mostly on the curved beam ; this will be discussed later.

The dam shown in Fig. 7 was designed with varying radii to keep the central angle of the arch as nearly constant as possible at all elevations. For comparison, a section is shown in Fig. 8, using the same unit compression (except where it is wider than a gravity section near the foundation) and the same up-stream face batter, but a single common center for both up-stream and down-stream faces. For this section the length of the up-stream radius is also variable, but it increases toward the bottom and reaches here a value of 322 ft.

The average vertical compression on the foundation for the section shown in Fig. 8 is $\frac{2,3}{2.2} H = 1.04 \times H$, and the corresponding total initial axial compression due to the lateral deformation is

$$\frac{1}{5} (1.04 \times H + H) = 0.41 \times H.$$

The height of water which this initial stress can resist, therefore, is equal to (see Formula 7)

$$0.41 \times H \frac{160}{322} = 0.20 \times H,$$

or 20% of the total head. Comparing this with 58.5%, or 42.2% for the constant-angle arch type (Fig. 7), it is easily seen that this latter type is much more effective in utilizing the initial stresses to support the water load than the ordinary arch having its faces struck from a single center.

If a gravity section is insisted on for the arch, but the central angle is kept as nearly constant as practicable, it will be possible for the gravity section to take up the greater part of the load, acting as an arch and curved beam toward the bottom. The factor of safety has thereby been increased several times with only a slight increase in material due to the smaller average radius used in the construction of the constant-angle arch.

It may be argued that shrinkage of the concrete while setting will compensate, to a large extent, for the axial expansion. This is true when the reservoir is empty and the concrete dry, but this condition is not of nearly as much interest or importance as that of reservoir full, for which condition it is not true. In this latter case the concrete is water-soaked, and in that state the length of the arch is at least equal to the original length before setting.* It has been shown in actual practice, however, that some concrete does shrink considerably, opening up contraction joints and cracks. One way of compensating for this, is to force grout into these contraction joints, at a time when the temperature is low and the reservoir empty, after having been filled once. In this way, the permanent set of the structure will also be compensated for, and the arch and cantilever will take their respective shares of the load when the reservoir is filled the next time, and each time thereafter.

Formula 5 has been used for finding the deflection curves, A and B, of the two sections, Fig. 7 (base 110 ft.) and Fig. 8. Formula 7 has been used for correcting these curves, A and B, to take the effect of lateral strain into consideration. These curves are shown in Fig. 9, and represent the deflections of the two arches, assuming that they

* See tests reported by Messrs. A. T. Goldbeck and A. H. White at the 1911 Convention of the American Society for Testing Materials.

are free to move at the foundation. They are plotted to show how evenly the deflection curve, *A*, slants from a maximum near the top (above Elevation 40 the two arches are identical, the "horns" being two tangents 40 ft. high) to nearly nothing at the bottom in the constant-angle arch type (Fig. 7), and how little the slant curve, *B*, amounts to in the ordinary type of arch dam (Fig. 8). These curves also show very plainly that from the common arch type much arch action toward the bottom cannot be expected; cantilever and beam action must take the load, because no such deflection as 0.2624 in. could be possible at the point where the arch is fastened to the rock foundation. The constant-angle arch type for this particular site, requiring only 0.0083 in. deflection, or 31.5 times less, to support the same load, will take most of the load on itself, acting as an arch.

For dam sites where the abutments are close to one another toward the foundation, and where t is large compared with R_u, Formula 5 gives values for the crown deflection which are too large, even assuming that the dam is entirely free to move at the bottom. Though this formula considers the curved-beam action, it is at the same time understood that arch action is complete. However, where the arch is thick and the distance between the abutments is short, the arch becomes a wedge, and the horizontal curved beam takes the greater proportion of the load, because, acting in this manner, the support of the same load will require a smaller deflection.

The deflection in the middle of a beam 1 ft. wide, held at both ends, and uniformly loaded, is

$$D_b = \frac{P \times l^4}{E\,J \times 384} \qquad (5a)$$

The notations are the same as before, P being the water pressure, in pounds per square foot; l, the length of the span, in feet; E, the modulus of elasticity of concrete per square foot; and J, the moment of inertia.

Whenever Formula 5*a* gives smaller values than Formula 5, it is indicated that arch action is incomplete. The curved-beam action tends to introduce axial tension along the down-stream face in the middle, and along the up-stream face near the abutments, but the axial compression due to the partial arch action and lateral expansion (Poisson's ratio) will, or should much more than, compensate for this tendency. If it does not, the design should be changed.

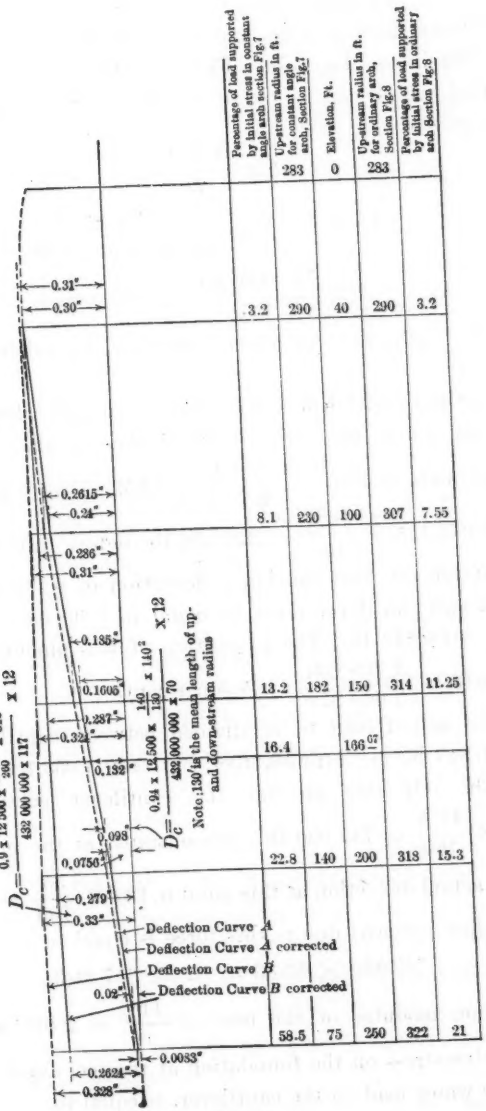

Percentage of load supported by initial stress in constant angle arch section Fig.7
Up-stream radius in ft. for constant angle arch, Section Fig.7
Elevation. Ft.
Up-stream radius in ft. for ordinary arch, Section Fig.8
Percentage of load supported by initial stress in ordinary arch Section Fig.8
283
0
283
0.31"
0.30"
3.2
290
40
290
3.2
0.2615"
0.24"
8.1
230
100
307
7.55
0.286"
0.31"
0.135"
0.1605"
13.2
182
150
314
11.25
0.287"
0.324"
16.4
166.07
0.132"
0.098"
0.0756"
22.8
140
200
313
15.3
0.279"
0.33"
0.092"
Deflection Curve A
Deflection Curve A corrected
Deflection Curve B
Deflection Curve B corrected
58.5
75
250
322
21
0.0083"
0.2624"
0.328"
$D_c = \frac{0.9 \times 12\,500 \times \frac{313}{260} \times 313^2}{432\,000\,000 \times 117} \times 12$
$D_c = \frac{0.94 \times 12\,500 \times \frac{140}{130} \times 110^2}{432\,000\,000 \times 70} \times 12$
Note :130/ft the mean length of up- and down-stream radius

FIG. 9.

From Curve *A* the deflection of the arch, Fig. 7, at the one-third point can be ascertained directly, and is found to be 0.132 in. If the cantilever, 250 ft. high and 1 ft. wide, were actually forced to deflect 0.132 in. at this point (Elevation 166.67 ft.) a force, *F*, would be required, which can be found as follows (*F* is concentrated at the one-third point):

$$D_c = \frac{8 \times F \times l^3}{E \times h^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$D_c = \frac{0.132}{12} = \frac{8 \times F \times 83.33^3}{432\,000\,000 \times 110^3},$$

$$F = \frac{759\,000\,000}{555} = 1\,368\,000 \text{ lb.}$$

(Where no explanation is given, formulas are taken from standard handbooks.)

The cantilever will deflect the same as the arch when thus loaded.

The total water load on a vertical slice of the dam, 1 ft. wide and 250 ft. high, is $250 \frac{0. + 15\,625}{2} = 1\,953\,125$ lb. The initial stress supports $1\,953\,125 \times \frac{16.4}{100} = 320\,312$ lb. before any deflection takes place, therefore the load causing a deflection of 0.132 in. of the combined arch and cantilever must be equal to $1\,368\,000 + 1\,953\,125 - 320\,312 = 3\,000\,813$ lb. The proportion of this amount taken by the cantilever will be $\frac{1\,368\,000}{3\,000\,813} = 45.5$ per cent.

Now, the actual load to be divided between cantilever and arch is not 3 000 813 lb. per running foot, but only $1\,953\,123 - 320\,312 = 1\,632\,813$ lb. Of this amount the cantilever carries 45.5%, or $1\,632\,813 \times \frac{45.5}{100} = 742\,930$ lb., concentrated at the one-third point, making the actual deflection at this point $0.132 \times \frac{55}{100} = 0.073$ in.

The bending moment due to this force is equal to

$$742\,930 \times 83.33 = 61\,908\,357 \text{ ft-lb.}$$

The section modulus of the base $= \frac{110^2}{6} = 2\,011$, and therefore the compressive stress on the foundation at the toe, due to the bending action of the water load on the cantilever, is equal to

$$\frac{\text{Bending moment}}{\text{Section modulus}} = \frac{61\,908\,357}{2\,011} = 30\,780 \text{ lb. per sq. ft.} \quad \ldots\ldots (9)$$

The total compression on the foundation at the toe will be this compression added to that due to the weight of the structure, which amounts to approximately 16 200 lb. per sq. ft. at the toe, making the total compression approximately 47 000 lb. per sq. ft.

If a base length of 70 ft. is chosen, the arch would take a greater percentage of the load and the curved beam a smaller, leaving the same or less for the cantilever, but, owing to the smaller section modulus of the 70-ft. base, the compression at the toe would be somewhat higher than 30 780 lb. per sq. ft., and the compression due to the weight of the structure would be much higher than 16 200 lb. per sq. ft., so that the sum of the two would be considerably more than 47 000 lb. per sq. ft. Although within the safe limit, the resulting vertical compression would be out of proportion to the 36 000 lb. per sq. ft. (and less) axial compression used when calculating t from Formula 1.

The dam section with the 110-ft. base contains only 4% more material than the dam with the 70-ft. base (Fig. 7), as the addition is not made as a portion of a circular ring, but in the shape of a spherical triangle, as indicated in Figs. 4 and 5. Any intermediate base length between the two limits given in Fig. 7 could be accepted for a dam built on this particular site. (This is shown approximately in Fig. 3, but there the tangents are not indicated.) The two stresses, the 36 000 lb. per sq. ft. average axial compression, and the maximum 47 000 lb. per sq. ft. vertical compression, are acting in planes perpendicular to each other, and therefore tend to support each other. Although they are low, the resulting section (Fig. 7) appears unusually slender on account of the economical distribution of the material.

This method of calculating the vertical stress on the foundation is correct only so long as no tension exists at the heel, or, if tension exists, so long as this tension is properly taken care of. For the constant-angle arch, where the cantilever takes the smaller proportion of the load, there will seldom be occasion for tension along the up-stream face, and there will perhaps never be enough tension to demand consideration. The accuracy of the result obtained from the use of Formula 8 depends to some extent on the face slopes, especially that of the down-stream face. The error, however, is generally such as to compensate for that made in not considering that the width of the

vertical cantilever, which is 1 ft. at the up-stream face, is less at the down-stream face. The short-cut method explained above for finding the division of the water load between cantilever and arch action and that for finding the total maximum foundation pressure, cannot be used for dams having a crown deflection curve similar to Line *B*, as this line does not show a maximum deflection near the crest and a zero deflection at the foundation. The deflection curve, *A*, answers these conditions closely enough for this purpose.

Thus far, only the middle or highest dam section has been considered, as we have been mostly interested in knowing the maximum stresses in the structure, which stresses generally occur (in high dams at least) at the toe, with reservoir full. Some modifications may be necessary near the abutments where the cantilevers, unless very short, do not support their share of the load, most of which is thrown over to the horizontal curved beam and arch, increasing the axial compression on the down-stream face from the abutments to somewhere beyond the points of contrary flexure. These points are located on both sides of the crown, and can be found from,

$$\cos.\ \theta_0 = \frac{\sin.\ \theta}{\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \ldots.(10)$$

as shown by Fig. 1. Reference should also be made to the discussion on "Lake Cheesman Dam and Reservoir".* This higher axial compression will not be entirely local, but will be transmitted from abutment to abutment through the middle of the dam, making the axial compression in the middle higher than that due to the proportionate water load carried by arch action; although still lower than the value obtained from Formula 1, assuming the whole load to be carried by the arch. The actual value may be found by drawing a graphical stress diagram, such as used for finding stresses in arch ribs for concrete bridges, etc., keeping in mind that, in the case of a dam, the directions of the forces are radial. From information as to the distribution of the axial stresses thus obtained, a new value for the deflection at the one-third point can be ascertained, and some readjustment between arch and cantilever action can be made. As long as the unit compression used is not much greater than 50 000 lb. per sq. ft., this refinement seems to be unnecessary, and hardly worth the additional labor.

* *Transactions*, Am. Soc. C. E., Vol, LIII, p. 167.

In Formula 1 only average stresses have been considered in determining the thickness of each individual arch slice. The maximum axial stresses should also be investigated. These exist along the down-stream face, and are found from the formula,

$$Q \text{ (max.)} = q \times \frac{2 R_u}{R_u + R_d} \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Formula 11, however, does not give correct results toward the foundation, where the arch is thick relative to the length of the up-stream radius, and where the span is short. The proportion of the load carried by the arch in such a case is supported more by the curved-beam than by ordinary arch action. This will cause some difference in the value of Q (max.) and q (min.) (as found from Formula 11), adding to Q (max.) at and toward the abutments, and subtracting from it in the middle portion between the points of contra-flexure on the curved beam. In high dams, Q (max.) will ordinarily be lower than the vertical compression at the toe, therefore this vertical pressure is still the most important to investigate.

The influence of Poisson's ratio tends to equalize Q (max.) and q (min.) in dam sections having up-stream faces of steeper slope than their down-stream faces. In such sections the vertical pressure due to the weight of material above is greatest along the up-stream face, and therefore the initial axial compression is also greatest. It is fair to assume that this condition of relieving Q (max.) and adding to q (min.) also tends to improve the water-tightness of the dam.

In all straight gravity dams built across narrow canyons, horizontal tension exists along the down-stream face in the middle, and along the up-stream face near the abutments, at least toward the foundation. This should be very plain when it is considered that any beam fixed at both ends and uniformly loaded will support four times as much load as a cantilever of the same length sustaining the same water load (nothing at the top and a maximum at the bottom). In other words, whenever the beam is four times longer than the cantilever, it will support half of the total load, and whenever this ratio is less than four, the horizontal beam will support most of the load. The ordinary gravity design does not consider this beam action, although, when the dam is built in a fairly narrow canyon, the greatest portion of the load toward the foundation is actually carried on the

horizontal beam, and not on the cantilever. Though adding materially to the stability of the dam (as long as the horizontal tension introduced by this beam action is not greater than the breaking point, and as long as the expansion joints, if any, are placed at or near the points of contra-flexure), the foundation pressure at the toe is at the same time much relieved, a very welcome feature, especially in connection with high dams, and surely this feature should not be left out of consideration when calculating the factor of safety.

Now, if the horizontal beam is curved, horizontal axial compression takes place over the entire section, and the greater the curvature (that is, the smaller the up-stream radius) the more load will be taken by the arch, and the less remains to cause horizontal axial tension at any point of the dam faces, due to beam action. The resultant axial compression from arch action and lateral expansion will in general more than compensate for this tension. Lateral expansion due to the weight of the structure exists, of course, whether the dam is straight in plan or curved, but this alone will seldom be sufficient to compensate for the horizontal tension due to beam action in a straight gravity dam across a narrow canyon. The curvature must be introduced in order to be sure that no tension exists in this horizontal beam. For a dam 250 ft. high, the bottom width of the canyon would have to be well toward ½ mile before a gravity dam would act simply as a gravity section toward the bottom, and before the influence of the horizontal beam action would be negligible, unless it should have failed in tension first. (Near the top, the horizontal beam would have no practical influence.) It would seem logical, therefore, to provide even quite long dams with a curvature sufficient to take care of the horizontal tension. Such dams would not be true arch dams, but the slight curvature would increase the factor of safety by eliminating the tension in the horizontal beam; and combined beam, cantilever, and some arch action would support the load.

Whenever a load is supported on a beam, or on a cantilever, shearing stresses are introduced. These shearing stresses reach their maximum values at the foundation and at the abutments, and should be investigated in order to be sure that they are within safe limits. In the case of the dam section shown in Fig. 7, base 70 ft., it can easily be shown that, even should the shear on the lower 50 ft. of the dam

correspond to the full water pressure, this stress would be entirely within the safe limit, amounting to approximately 4 000 lb. per sq. ft. where the dam joins the hillside and foundation. Along this joint the maximum unit compression generally exists. This compression is so much larger than the shear that actual shear cannot take place, as friction alone will prevent any tendency to sliding at the abutments. Crushing would have to take place before the actual sliding of any element could occur. A fair approximation of the magnitude of the actual shear forces along the joint between the dam foundation and abutments can be obtained in the following manner: Formulas 5 and 5*a* are both applied in order to find in what proportion the load is divided between arch action and curved-beam action toward the bottom. Suppose Formula 5*a* gives, say, half as much deflection as Formula 5 for the same load; then it is indicated that the curved beam carries twice as much load as the arch. In reference to this the factor, CC_c, in Formula 5 should be left out, as we are working close to the foundation, where the arch is short and thick, and where it is therefore necessary to use both Formulas 5 and 5*a*.

From Formula 7 the percentage of the total load carried by the initial axial stresses can be found, and from Formula 8, etc., the percentage of the remaining load (due to head, $H - h$) taken by the cantilever, can be determined. In the case of the dam shown in Fig. 7, this percentage was found to be 45.5, and there is, therefore, 54.5% (of $H - h$) left for the arch and curved beam. Out of this the arch takes $\frac{1}{3} \times 54.5\%$, and the curved beam $\frac{2}{3} \times 54.5\%$ if Formula 5*a* gives half as much deflection as Formula 5. The shear is caused by a force equal to $45.5 + \frac{2}{3} \times 54.5\%$ of the load that remains after that carried by the initial axial stresses (Poisson's ratio) has been subtracted from the total load. Another place where shear action exists is in a vertical plane at the foundation along the toe when the reservoir is full of water. The tipping in a down-stream direction of the loaded cantilever deforms the toe and forces it to make a depression in the foundation, and, as the rock foundation extends outside the concrete toe, shear will take place where the end of the toe tries to deform the foundation. This circumstance introduces additional foundation

pressure at the toe, but this increase is already included in the value as found from Formula 9, at least approximately.*

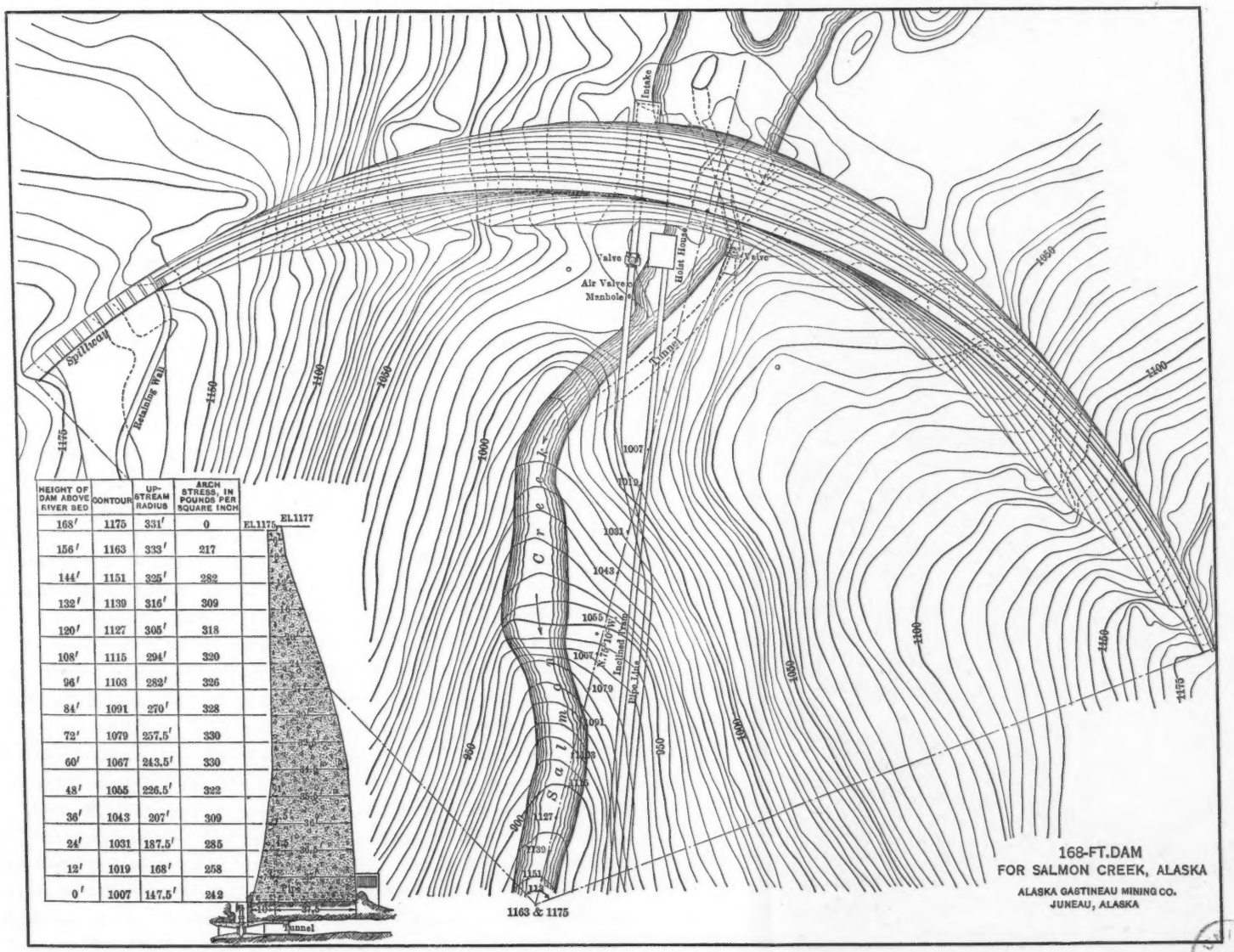
Before concluding the general discussion, the action of changes in temperature should be considered. As the ends of the arch are fixed to the abutments, the shortening or lengthening of the arch due to temperature changes either causes cracks to develop or forces the crown back and forth. In the constant-angle arch type the average crown deflection for the same amount of decrease or increase in length of arch is only about half of that required by the common arch type (see Curves *A* and *B*, Fig. 9), and the tension necessary to cause this deflection or pulling back of the cantilever, therefore, may not exceed the ultimate strength of the concrete, in which case no cracks would develop. In any event, cracks are not as likely to occur as in the common arch type of dam.

Practical Examples of Arch Dams.

Plates IX and X show the plan and section of the Salmon Creek Dam, 168 ft. high above the river surface, containing 52 000 cu. yd. of concrete, having 1.25 bbl. of cement and a small percentage of lime per cubic yard. The contours represent the actual condition of the site after excavation, and it is seen that the sides of the canyon form an unusually regular V. The crest width of the dam is approximately 550 ft., measured in a straight line, and the arch at this elevation subtends an angle of 113 degrees. Plate IX shows the location of the centers for various arch slices, 12 ft. apart in elevation, and the length of the corresponding up-stream radii and unit axial stresses are given in the table on Plate X. To provide better accommodation for the spillway, the curve for the top 12 ft. of the dam was struck from the same center, therefore the warping of the faces commences 12 ft. below the crest, and continues down to the foundation. This warping is so uniform that one who does not know that the centers are moved constantly in an up-stream direction toward lower elevations does not notice it. The carpenter gets his points about every 10 ft. apart, and it makes no difference to him whether he builds up the

*The result from Formula 9 depends on Formula 8, and in Formula 8 the deflection due to the bending only is considered, not the additional deflection due to the horizontal shear forces accompanying the bending action, which amount to 10% (average), therefore the proportion of load taken by the cantilever, as given on page 1302, is really 10% too high, but this makes the resulting foundation pressure at the toe, found from Formula 9, more nearly correct on account of the action of the vertical shear at the toe, and this is the information that concerns the designer most.

PLATE IX.
PAPERS, AM. SOC. C. E.
MAY, 1914.
JORGENSEN ON
THE CONSTANT-ANGLE ARCH DAM.



HEIGHT OF DAM ABOVE RIVER BED	CONTOUR	UP-STREAM RADIUS	ARCH STRESS, IN POUNDS PER SQUARE INCH
168′	1175	331′	0
156′	1163	333′	217
144′	1151	325′	282
132′	1139	316′	309
120′	1127	305′	318
108′	1115	294′	320
96′	1103	282′	326
84′	1091	270′	328
72′	1079	257.5′	330
60′	1067	243.5′	330
48′	1055	226.5′	322
36′	1043	207′	309
24′	1031	187.5′	285
12′	1019	168′	258
0′	1007	147.5′	242

face of a cylinder or an inverted cone (approximately). The surveyor, however, has to be more careful than with the layout of an ordinary arch, as, in the present case, there are more calculations to be made and to be followed.

The table on Plate X shows that the length of the longest up-stream radius is 333 ft., and the length of the shortest 147.5 ft., the ratio between the two being $\frac{333}{147.5} = 2.26$.

Had the length of the up-stream radius been kept constant, the thickness of the dam at the bottom would have had to be increased 2.26 times for the same axial stresses. Relative to this it should be noted that the arch stresses in the table assume the arch to take the total load, but in reality the stresses are somewhat smaller, as the cantilever takes part of the load. The triangular piece, 10 ft. wide at the bottom (which is added to the lower part of the dam on the down-stream side for the purpose of stiffening the cantilever where it is highest), is not considered in the table giving the arch stresses.

To have kept the enclosed angle (113°) constant at all elevations would have necessitated a greater ratio than 2.26 between the length of the two up-stream radii already referred to. Had this ratio been increased, the structure would have been overhanging in places, and therefore this increase could not be made. This simply shows that it is not always possible to make theory and practice coincide exactly.

To have kept the subtended angle constant in this case would have necessitated a greater bottom width of the site, other conditions remaining the same, but, of course, the dam has to be fitted to the site, and not *vice versa*. This dam creates a reservoir having a capacity of 826 000 000 cu. ft. The drainage area is only 7.5 sq. miles, but the precipitation makes up for it, being more than 100 in. per year.

Plate X shows a plan and section of the construction plant, and is self-explanatory. This construction plant, which proved to be the best kind that could be used for this particular place, was laid out under the supervision of Mr. H. L. Wollenberg, who was Chief Engineer of the Alaska Gastineau Mining Company, Juneau, Alaska, the owners, and had charge of this and other work. F. G. Baum, Assoc. M. Am. Soc. C. E., was Consulting Engineer for the Mining Company. The original design of the dam was made by the writer. Plates IX and X, however, were made by the Gastineau Mining Company.

F. C. Herrmann, M. Am. Soc. C. E., Chief Engineer of the Spring Valley Water Company, San Francisco, A. P. Davis, M. Am. Soc. C. E., Chief Engineer of United States Reclamation Service, and the writer visited and reported on the project at various times, and Mr. Herrmann wrote an extensive report thereon.

Figs. 10 to 13, inclusive, show different construction features, thus: The double tower with the two hoppers from which the two concrete chutes distribute the concrete over the dam. These chutes are supported on triangular wooden towers, easily removable. Two ¾-cu. yd. Smith mixers were used, and the progress was at the rate of about 400 cu. yd. daily. Work was shut down for the winter about November 1st, 1913, at which time the dam was half completed. It is expected to have it finished by August 1st, 1914. Indications are that the unit cost will be $7.50, or slightly less, including everything. The electric power plant, depending on the reservoir for its steady supply, is now in operation. Mr. D. C. Jackling is Vice-President, in charge of the operations of the Gastineau Mining Company, and Mr. B. L. Thane is Manager, with offices in Juneau, Alaska.

Plate XI shows the constant-angle arch principle applied on the Lake Spaulding Dam. This dam is part of the South Yuba development, undertaken and owned by the Pacific Gas and Electric Company, San Francisco, Cal.* The lower portion of this dam (60 ft.) is provided with a gravity section for a 260-ft. head, and is arched in plan, the length of the up-stream radius being 600 ft. up to Elevation 4 628. When this elevation was reached (less at the down-stream face, as shown on Plate XI) work was shut down for the winter, 1912-13. During the winter the original plans were changed, and in the following summer the dam was continued to Elevation 4 825, according to the plans on Plate XI. At Elevation 4 628 the length of the up-stream radius was changed from 600 ft. to 250 ft. and kept at this length up to Elevation 4 675. From this level up to the crest, the length of the up-stream radius increases so as to keep the subtended central angle as constant as possible, as shown by the table on Plate XI. This subtended angle is not as large as could be desired, but is as great as the

*This development has been described in detail in articles in *Engineering News*, *Engineering Record*, and others during 1913. Probably the most complete and accurate article on the subject is that written by R. W. Van Norden, M. Am. Soc. C. E., in the *Journal of Electricity, Power and Gas*, San Francisco, December 13th, 1913.

PLATE X.
PAPERS, AM. SOC. C. E.
MAY, 1914.
JORGENSEN ON
THE CONSTANT-ANGLE ARCH DAM.

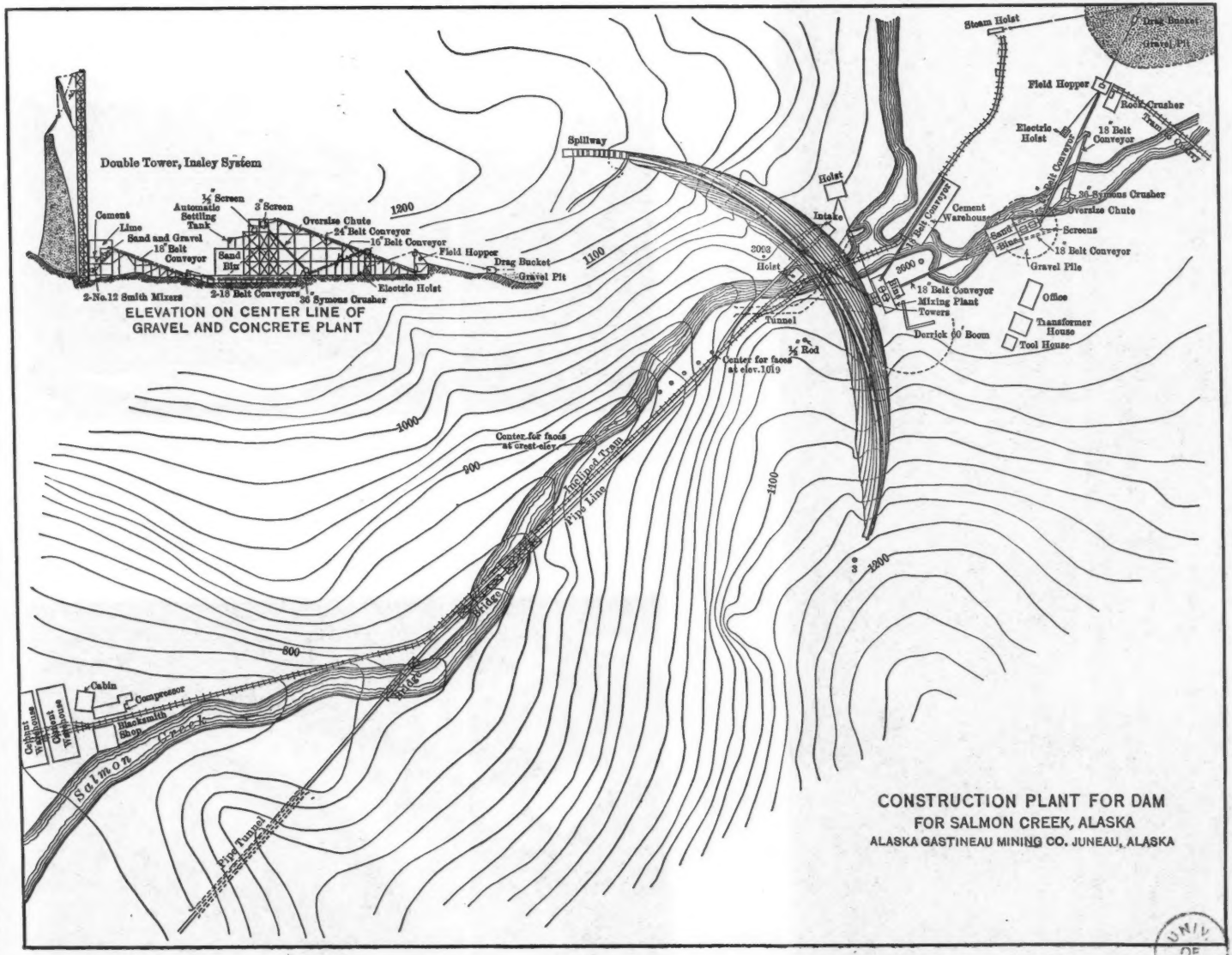


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FIG. 10.—RESERVOIR SITE ON SALMON CREEK, ALASKA. LOOKING UP STREAM FROM THE DAM. GRAVEL PITS IN THE FOREGROUND.

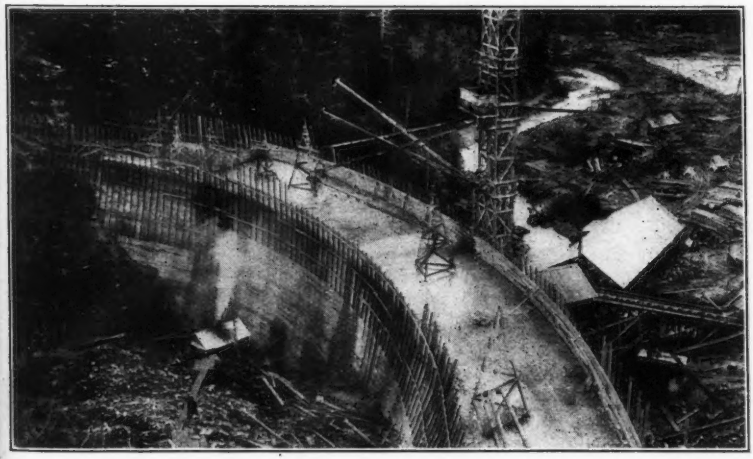


FIG. 11.—SALMON CREEK DAM, SHOWING CONSTRUCTION TOWER AND FORM WORK.

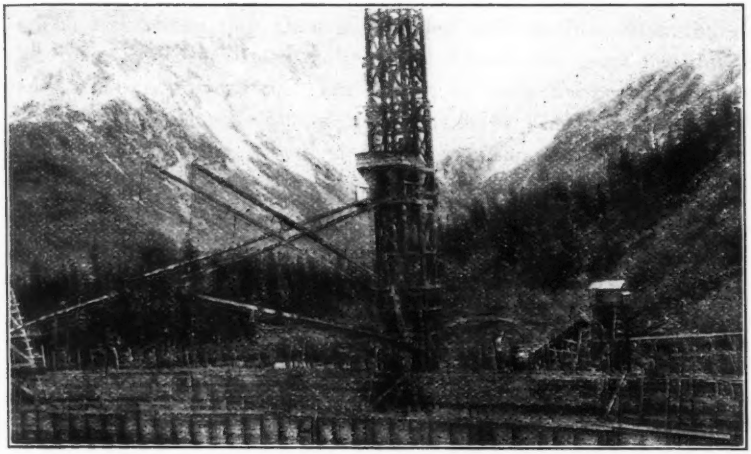


FIG. 12.—SALMON CREEK DAM. DETAILS OF TOWER AND DISTRIBUTING SYSTEM FOR CONCRETE.

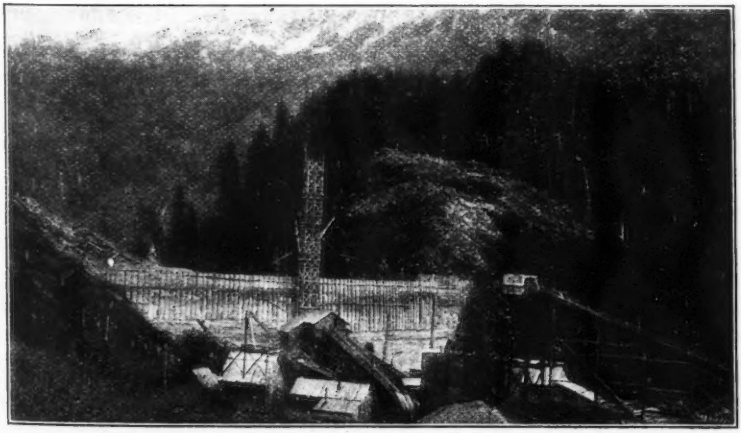


FIG. 13.—SALMON CREEK DAM. CONSTRUCTION PLANT.

site would permit, considering that the ultimate proposed crest elevation is to be at Elevation 4 905.

The Lake Spaulding Dam is provided with an inspection tunnel, a drainage system, and contraction joints, which are usual features in dams of large proportions. The details of these features are shown on Plate XI. The section of the arch above Elevation 4 660 is of such dimensions that it will stand an extension of 35 ft. in height above the present crest elevation (4 825) without any addition to its thickness. The maximum arch stress (q in Formula 1) will exist at Elevation 4 775 with the water level at Elevation 4 860, or 260 ft. above the river bed, and will amount to 23.8 tons. It is fairly constant over the greatest portion of the structure, as can be seen from the table on Plate XI.

When the time comes to extend the crest of the dam to Elevation 4 905, 305 ft. above the river bed, a slab of concrete must be added to the down-stream face, and, in order to effect a good bond between the present dam and the new slab, the down-stream face of the present dam has been stepped off and a sufficient number of iron rods have been left protruding several feet to grip the new slab and hold it in place.

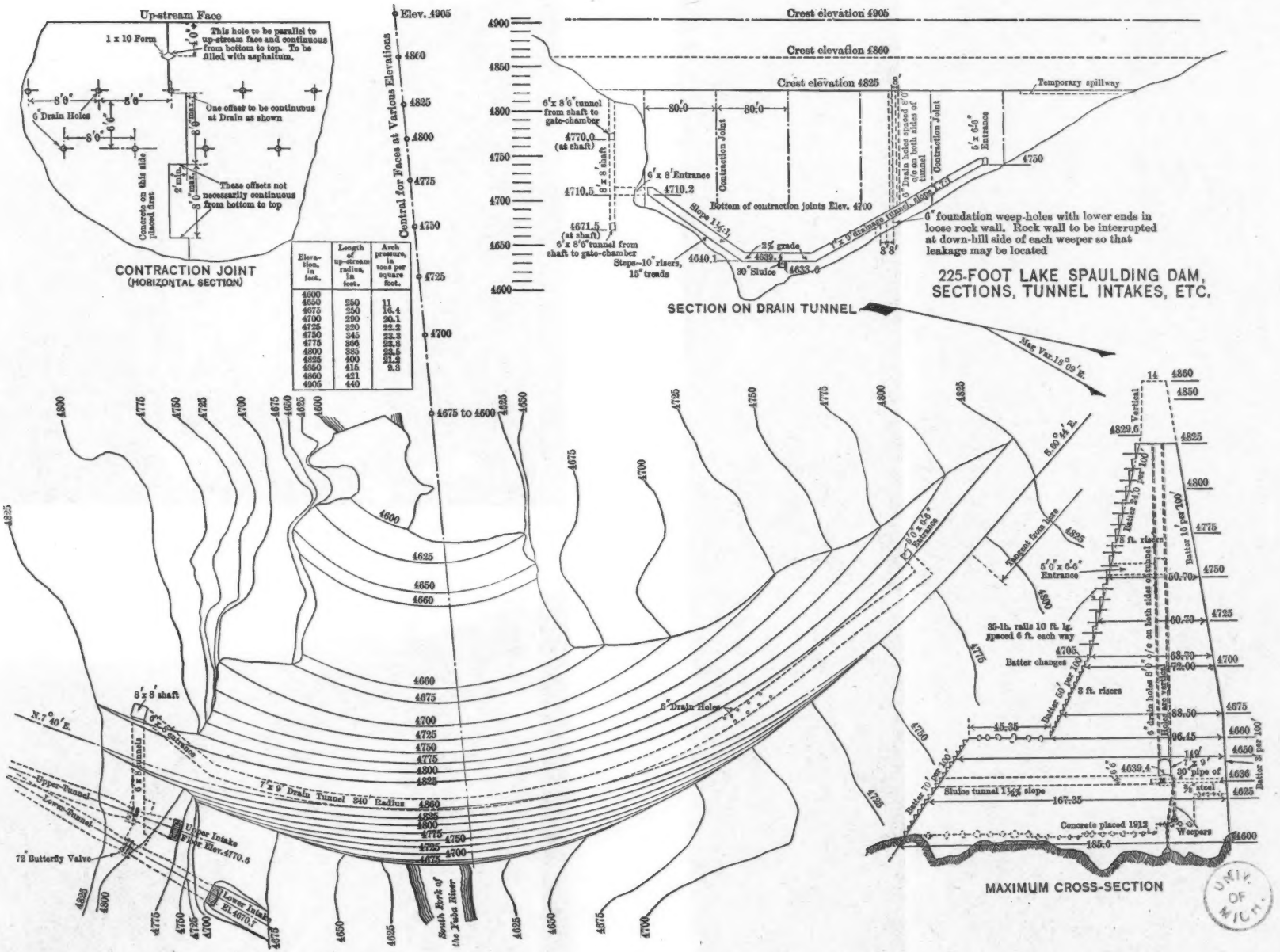
The Lake Spaulding Dam was constructed of a wet mixture of gravel concrete. Toward the top this mixture was richer than at the bottom, and contained from 1.17 to 1.25 bbl. of cement per cubic yard. Test specimens were broken every day, and the results of these tests were used as a guide in fixing the exact proportion of the mix. It was required that the breaking strength of the test specimens (8 in. in diameter) should be at the rate of 400 lb. per sq. in. when 7 days old; and 900 lb. per sq. in. when 28 days old. Toward the lower elevations, where the arch was thick, the mixture was somewhat leaner. The concrete was mixed in four 1-cu. yd. Smith mixers on the hillside above the dam, and, after being turned in the mixers for 1½ min., it was run by gravity in flumes to different portions of the dam, as shown on Figs. 14 and 15. After the dam had risen to a certain height the concrete could flow no more by gravity to portions farthest away, and a 30-in. belt-conveyor system was installed. With this system, all portions of the dam could be reached. There were also two cableways, to transport lumber and other materials to the dam. During the latter part of 1913 there was

very little water in the reservoir, the outside temperature was low, and the chemical heat was practically out of the dam body. This had the effect of opening up the contraction joints (which are 80 ft. apart) about $\frac{1}{8}$ in. In addition to this, cracks about $\frac{1}{16}$ in. wide, appeared midway between each contraction joint. These were all along radial lines, the same as the contraction joints. When the water in the reservoir rose to the crest, early in February, 1914, these cracks closed.

The outlet works from the reservoir consist of two intake tunnels (shown on Plate XI), concrete lined, and of a finished diameter of 8 ft. 8 in. One intake is at Elevation 4 670; the other is 100 ft. above. Each intake is provided with a 72-in. butterfly valve. The upper intake slopes downward about 48° until it meets the lower tunnel, this slope starting a few feet back of the upper butterfly valve. About 1 000 ft. down stream the single pressure tunnel ends in an adit, and is there provided with a second butterfly valve and also with two pressure reducers. Later, it is intended to install a 5 000-kw. turbine and let this act as a pressure reducer by utilizing whatever head there may be in the reservoir. From this point the 350 sec-ft. of water will flow by gravity toward the power-houses below, of which one is built, and four more are projected. After the water has left the lowest power-house, it will be used for irrigation.

The actual construction work was started under the direction of the late James H. Wise, Assoc. M. Am. Soc. C. E. It was continued and completed to date with F. G. Baum, Assoc. M. Am. Soc. C. E., as Chief Engineer. John R. Freeman, A. P. Davis, and H. F. A. Schussler, Members, Am. Soc. C. E., and others acted as Consulting Engineers at different times. Mr. Freeman, working in conjunction with Mr. Davis, made an extensive report, and many of the details suggested by these gentlemen are incorporated in the construction of the dam. Mr. Davis visited the site several times during the construction period. In the actual design as used, the writer's arch theory, and the shape of the up-stream face suggested in designs submitted by him, have been followed, except near the foundation, where an ordinary conical face was substituted below Elevation 4 675, instead of continuing the inverted cone (approximately) above.

Mr. John A. Britton was General Manager and Vice-President of the Company, H. C. Vensano, Assoc. M. Am. Soc. C. E., Civil Engineer. R. G. Clifford, Assoc. M. Am. Soc. C. E., Designing Engineer


Up-stream Face
1 x 10 Form
This hole to be parallel to up-stream face and continuous from bottom to top. To be filled with asphaltum.
6" Drain Holes
8'0"
8'0"
8'0"
One offset to be continuous at Drain as shown
These offsets not necessarily continuous from bottom to top
Concrete on this side placed first
CONTRACTION JOINT
(HORIZONTAL SECTION)
Elev. 4905
4860
4825
4800
4775
4750
4725
4700
4675 to 4600
Central for Faces at Various Elevations
Elevation, in feet. | Length of up-stream radius, in feet. | Arch pressure, in tons per square foot.
4600 | | 11
4650 | 250 | 16.4
4675 | 250 | 20.1
4700 | 290 | 22.2
4725 | 320 | 23.3
4750 | 345 | 23.5
4775 | 366 | 23.6
4800 | 385 | 21.2
4825 | 400 | 9.8
4850 | 415 |
4860 | 421 |
4905 | 440 |
Crest elevation 4905
Crest elevation 4860
Crest elevation 4825
Temporary spillway
6'x 8'6" tunnel from shaft to gate-chamber
4770.0 (at shaft)
6'x 8' shaft
4710.5
4671.5 (at shaft)
6'x 8'6" tunnel from shaft to gate-chamber
Steps-10" risers, 15" treads
80'0
80'0
Contraction Joint
Contraction Joint
6'x 8' Entrance
4710.2
Bottom of contraction joints Elev. 4700
Slope 1½:1
2% grade
4640.1
4639.4
30" Sluice
4633.6
7'x 9' drainage tunnel, slope 2%
6" Drain holes spaced 8'0" c/c on both sides of tunnel
6'x 6'6" Entrance
8'8'
6" foundation weep-holes with lower ends in loose rock wall. Rock wall to be located at down-hill side of each weeper so that leakage may be located
225-FOOT LAKE SPAULDING DAM,
SECTIONS, TUNNEL INTAKES, ETC.
SECTION ON DRAIN TUNNEL
Mag. Var. 18° 09' E.
8° 40' E.
Vertical
4829.6
Batter 2½ per 100'
Batter 10 per 100'
5 ft. risers
Tangent from slope
5'0"x 6'6" Entrance
50.70
60.70
68.70
72.00
35-lb. rails 10 ft. lg. spaced 6 ft. each way
Batter changes
3 ft. risers
88.50
96.45
149'
30" pipe of ¾" steel
Sluice tunnel 1½% slope
167.35
Batter 70' per 100'
Batter 8' per 100'
Concrete placed 1912
Weepers
185.6
MAXIMUM CROSS-SECTION
N. 7° 40' E.
8'x 8' shaft
6'x 6' tunnels
6'x 6' Entrance
7'x 9' Drain Tunnel 240' Radius
Upper Tunnel
Lower Tunnel
Upper Intake Floor Elev. 4770.6
72" Butterfly Valve
Lower Intake El. 4670.7
6" Drain Holes
South Fork of the Foks River

UNIV. OF MICH.

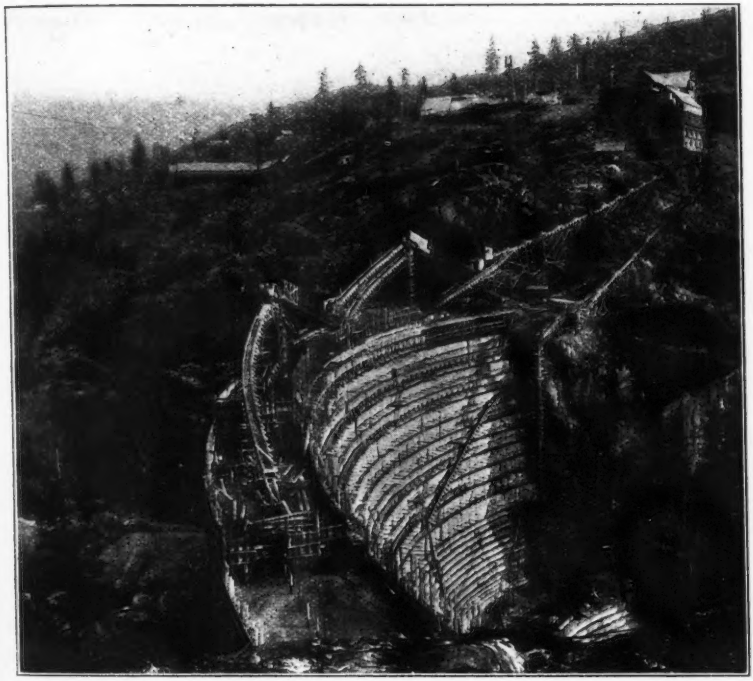


FIG. 14.—LAKE SPAULDING DAM. MIXING HOUSE, DISTRIBUTING FLUMES, AND BELT CONVEYORS. THE METHOD OF CONSTRUCTING THE EXPANSION JOINTS CAN BE SEEN.

FIG. 15.—DOWN-STREAM VIEW OF LAKE SPAULDING DAM, SHOWING THE STEPPING-OFF AND IRON RODS FOR BOND.

FIG. 16.—DOWN-STREAM FACE OF LAKE SPAULDING DAM, 225 FEET HIGH ABOVE RIVER BED.

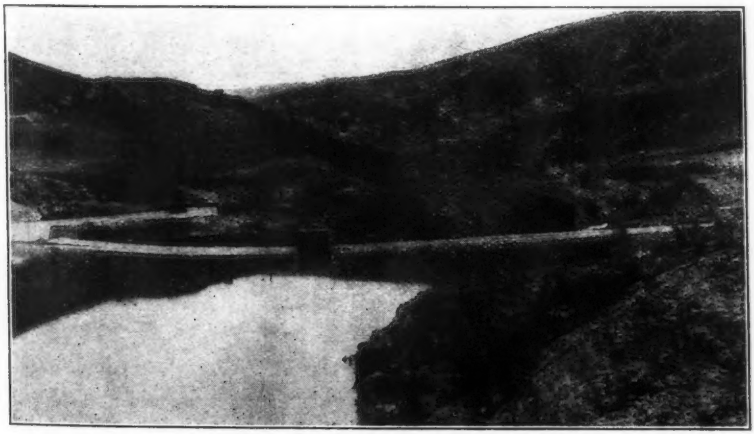


FIG. 17.—A SMALL OVERFLOW DAM. SHOWING OBSTRUCTION IN THE MIDDLE TO FACILITATE THE ADMISSION OF AIR BEHIND THE SHEET OF FALLING WATER.

and Resident Engineer when the dam was started. Mr. P. Magerstadt was Resident Engineer during 1913, and Messrs. Duncanson and Harrelson superintended the construction.

An arch dam 30 m. (98.4 ft.) high, with a base width of 4 m. (13.12 ft.) and a crest width of 1 m. (3.28 ft.), having an up-stream radius of 20 m. (65.6 ft.) at the bottom and 35 m. (114.8 ft.) at the crest, has been designed and built by H. F. Cameron, M. Am. Soc. C. E., Division Engineer of the Bureau of Public Works, Manila, P. I., to store water for domestic use.*

Fig. 17 shows a small diverting overflow dam designed on the constant-angle arch principle. It is built at an elevation of 13 000 ft. above sea level, in the Peruvian Andes, and is part of a hydraulic development by the Cerro de Pasco Mining Company. Messrs. F. G. Baum and Company, of San Francisco, were the engineers, and Mr. H. L. Wilcox was the Superintendent. This dam is designed to take care of a maximum overflow of 1 500 sec-ft., and is 62 ft. high, from bed-rock to crest, but only 24 ft. high above the river bed. The length of the up-stream radius at the crest is 36 ft., and at the river bed, 18 ft.

* *Engineering Record*, August 23d, 1913, p. 203.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SUBAQUEOUS HIGHWAY TUNNELS.

By George Duncan Snyder, M. Am. Soc. C. E.

To be Presented September 16th, 1914.

Synopsis.

The object of this paper is to call attention to the use of tunnels as a means of crossing rivers or bodies of water by highways, particularly when the waters are navigable, and to discuss the conditions where tunnels are most suitable and where not.

The relative advantage of ferries, movable bridges, swinging draw-, bascule-, lift-, and transporter-bridges, are discussed, and a table showing the general dimensions of the principal transporter-bridges, is given. The suitability of pontoon bridges is considered, and also the relative advantages of fixed bridges and tunnels, and there is a table showing the general dimensions and cost of the principal bridges over navigable waters. The paper contains a history and description of existing subaqueous highway tunnels, including the Thames, Blackwall, and Rotherhithe Tunnels, in London, the Washington Street, La Salle Street, and Van Buren Street Tunnels, in Chicago; the Glasgow Harbor Tunnel, and the Hamburg Tunnel; and a table gives their general dimensions.

Projects for subaqueous highway tunnels at New York, Boston, Chicago, Sydney, Liverpool, and Oakland, Cal., are considered. Methods of construction are discussed, which include rock tunnels,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

and, owing to their bearing on the subject, the Severn railway tunnel and the Mersey railway tunnel are described. Shield and compressed-air methods are described, as well as roof-shield, cut-and-cover, inside coffer-dam, and special methods, such as those used in the Harlem River crossing of the New York subway and the Detroit Tunnel. The caisson method used in Paris is considered, and also the freezing method.

Various types of lining—cast iron, brick, concrete, wrought steel, etc.—are described.

The shape of the section, means of access by inclines or shafts, methods of water-proofing, ventilation, and drainage, are also dealt with.

Conclusions.

The paper is a compilation of the available facts relative to subaqueous highway tunnels and other subaqueous tunnels applicable to highway purposes, and the conclusion is that, owing to the development of great harbors and the consequent need for unobstructed waterways and the increasing use of motor-driven vehicles, highway tunnels have not received sufficient consideration and are worthy of more extended discussion and attention.

Introduction.

With the growth of sea-borne commerce and its concentration at great ports, the problem of crossing harbors and their tributary navigable waters has become serious. The land traffic in the vicinity of a great harbor naturally develops with the water traffic, and inter-communication for land vehicles between the sides of a land-locked harbor is essential. The motor truck is being developed very rapidly and, if adequate facilities for inter-communication are provided, will develop still more rapidly and furnish a means of transferring goods from ocean vessels to railways, warehouses, and the like, thus relieving the small harbor craft and the railways from the burden of this transfer.

Where the harbor is not crowded, and with a low range of tide and a comparative freedom from fog and ice, a steam ferry is a fairly satisfactory means of crossing, as there are no excessive gradients of approach and the motors are idle or the horses resting while crossing. If, however, the harbor is crowded with water craft, there are two lines of traffic crossing each other's path on the same plane, or what in

railway parlance would be called a level or grade crossing, the avoidance of which would necessitate an under or over crossing, either by a tunnel or a bridge. With a great range of tide, it becomes difficult to load and unload boats, on account of the length of adjustable ramp necessary to avoid excessive gradients. With fog, ice, and bad weather, the danger of collision is increased, and the delays in loading, unloading, and operating ferries are serious, all of which add greatly to the cost of trucking.

On comparatively narrow channels, movable bridges of various sorts are used, but these do not avoid altogether the level crossing, as only the smaller craft can pass without having the draw-span opened. Many similar bridges are now being built at a level which will permit tugs and harbor craft to pass under them and thus decrease the frequency of opening. A clear space of 30 ft. from the surface of the water to the lowest member of a bridge will permit most harbor craft to pass under.

With a swinging draw, the large pivot pier required is a serious obstruction to the channel, and the maximum width of waterway, allowing for piers and fenders, is about 200 ft.* With a two-leaf bascule draw, the maximum span is about 300 ft. A lift-bridge has been built at Portland, Ore., with a span of 245 ft. and a clear headway of 135 ft. It will thus be seen that, at the best, such bridges interfere with the navigation of the waterway and fail to provide the free channels that an ocean port should have; also, the frequent opening of the draw obstructs the highway, and there is always the danger of a vessel colliding with a bridge.

Where the land traffic is light, transporter-bridges have been built. These are adapted to conditions where storms make the water very rough and cause difficulties in ferry navigation, or where the stream is obstructed at times by floating ice, or where a great fluctuation in the height of the water makes the design and construction of suitable landing ramps difficult. In general, they are erected at points where the importance of the river traffic is such that it cannot be hampered by a draw-bridge, where the traffic is so light as not to warrant the building of a fixed bridge or tunnel, or where the physical difficulties in the construction of a bridge or tunnel are very great.

* A swinging draw-bridge is to be constructed at Vancouver, B. C., with a length of 581½ ft. between end bearings; this will be the longest in existence and will have a clear width of waterway of 225 ft. (*Engineering News*, February 19th, 1914.)

Table 1 gives the general dimensions of the principal transporter-bridges.

TABLE 1.—TRANSPORTER-BRIDGES.

Location.	River.	Span, in feet.	Clear height, in feet.
Widnes and Runcorn, England	Mersey	1 000	82
Newport, England	Osk	645	177
Rouen, France	Seine	644.91	167.46
Middlesbrough, England	Tees	564.75	160.9
Marseilles, France	(Harbor)	541	164.3
Bilbao, Spain	Nevrion	524.96	147.65
Nantes, France	Loire	462.62	165
Martrou, France		448.55	164.05
Rochefort, France	Charente	415	165
Duluth, Minn., U. S. A.	(Harbor)	393.75	135
Kiel, Dockyards		387.2	145
Bizerte (near Tunis, Africa)		357.63	147.65
Brest, France	(Harbor)	357.6	144.4
Osten, Germany	Oste	262.5	83.8

It will be seen from Table 1 that the longest span is 1 000 ft., between Widnes and Runcorn,* over the Mersey, shown by Fig. 1. The only example in America is at Duluth, with a span of 393.75 ft.†

Most of the transporter-bridges in Table 1 are over harbors, canals, arms of the sea, or inland waters, and practically no through highway for ocean-going vessels is spanned by such a structure. There are many places where no other form of construction is warranted by the traffic; but, at the best, transporter-bridges are only suitable for comparatively narrow channels, where neither the water nor land traffic is heavy, and not for the crowded conditions of a great ocean port.

Pontoon bridges, with movable sections which can be floated to one side to permit the passing of boats, have been built. Such bridges are generally used in sparsely settled regions, where the land traffic is light; they are more or less temporary in character, and are a cheap substitute for a more permanent structure where the traffic does not warrant the heavier expenditure. However, a bridge of this type, crossing the Golden Horn, at Constantinople, which is more permanent in character, has recently been completed. The length between abutments is 1 530.84 ft., and the width of roadway, 82 ft. For small boats there are two spans of 39 ft. having a clear headway of 17½ ft., and a draw section with a clear opening of 205 ft.‡

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXV, p. 87.
† *Transactions*, Am. Soc. C. E., Vol. LV, p. 322.
‡ *The Engineer*, December 6th, 1912.

Numerous bridges of this type have been built in India.* In Calcutta one has been designed with submerged pontoons anchored at a fixed level so as to avoid the change in level of the roadway with the fluctuations of the water surface.†

Several pontoon railroad bridges of considerable length have been used in America. One, with a draw section 405 ft. long, was built at Prairie du Chien, Wis., in 1874, by the Chicago, Milwaukee and St. Paul Railway Company; this company also had four other bridges of similar type. These bridges, instead of being separate pontoons acting as piers, were really continuous floating structures.

Bridges of this type are not suited to a busy harbor frequented by numerous ocean-going vessels, and where there is a heavy land traffic.

The only remaining alternatives are fixed bridges at sufficient height to clear the shipping, or tunnels or passages at a greater depth than the draft of the vessels.

In avoiding the level crossing of these two conflicting kinds of traffic, by placing the roadway above or below the water, there is an advantage in depressing it, as the top of the structure need only be about 40 ft. below low water; whereas, if elevated, the bottom must be from 135 to 175 ft. above high water. This paper, however, is not written with the view of entering into a controversy as to the relative merits of bridges and tunnels, as no general law can

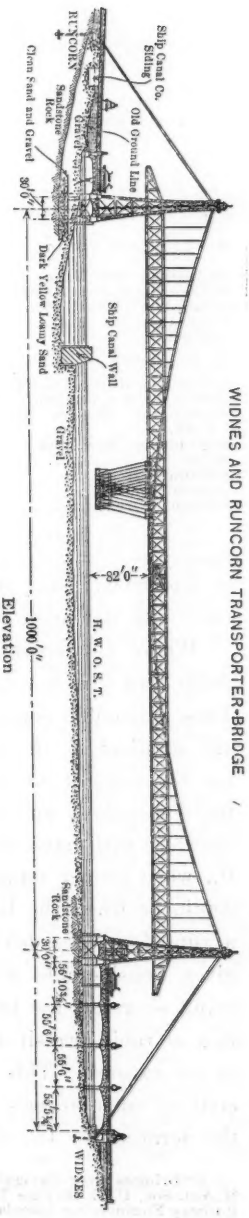


Fig. 1.

Elevation

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIX, p. 292.

† *The Engineer*, September 13th, 1912.

TABLE 2.—PRINCIPAL BRIDGES OF THE WORLD.

Location.	Character.	Length of longest span, in feet.	Total length of main spans, in feet.	Total length, including approaches, in feet.	Clear height above high water, in feet.
Forth	Cantilever	1 700	5 349.5	8 298.42	150
Brooklyn, New York	Suspension	1 595.5	3 455.5	7 580	135
Manhattan, New York	"	1 470	2 920	6 855	.....
Williamsburg, New York	"	1 600	2 793	7 308	135
Queensboro, New York	Cantilever	1 182	3 724.5	7 449	135
Poughkeepsie	"	548	3 093.75	6 777	130
Quebec	"	1 800	2 830	3 239	150
Tower, London	Bascule	200	880	2 680	29.5 / 143
Connecting R. R., Hell Gate	Steel arch	977.5	977.5	17 908	135
Clifton, Bristol, England	Chain suspension.	702.25	.....	.....	245
Washington Bridge, New York	Steel arch	510	1 660	2 375	133.5
High Bridge, New York	Stone arch	80	1 450	.....	100
Menai	Suspension	570	570	1 034.71	72
Britannia	Tubular.	459.25	1 513	1 841.42	103.75
Saltash-Tamar R.	Lenticular	436	910	2 240	100
Portland, Oregon	Lift	245	.....	.....	135
" "	"	220	.....	.....	144

be laid down; each problem must be decided on its merits in accordance with the conditions.*

Where the conditions are favorable for a bridge, there is no doubt that it is the most desirable means of crossing a body of water. These favorable conditions are high banks, which reduce the length and gradient of the approaches; moderate spans; a suitable stratum for the support of foundations at a reasonable depth; cheap land for approaches; and the ability to place piers so that they will not interfere with navigation. When the spans must be in excess of 1 000 ft., when proper support for foundations can only be had at a great depth, or when the land near the water is low, the conditions favor a tunnel. When bridges exceed a span of 1 000 ft., they must be given considerable width, for lateral stability, and the dead loads being so great, the bridges can be given great capacity by the addition of rapid transit and street railway tracks without adding greatly to the expense. This causes an undue concentration of traffic at the ends of such bridges and adds greatly to the difficulty and cost of the terminals. In other words, instead of building a bridge with

* "Bridges over Navigable Rivers—Some Practical Considerations," by C. E. Smith, M. Am. Soc. C. E., Bridge Engineer, Missouri Pacific Railway, *Proceedings*, American Railway Engineering Association, Vol. XIV, Part 2, p. 185.

OVER WATERS FREQUENTED BY SEAGOING VESSELS.

Width, in feet.	Use.	Dates.		Depth of water, in feet.	Cost.	Gradient of approaches.
		Work begun.	Work finished.			
30	Railway.	1883	1890	200	$15 700 000	1.43%
86	Highway, footway, electric railway, trolley.	Jan. 3, 1870	May 24, 1883	62	22 400 000	3.25%
120	"	Oct. 1, 1901	Dec. 31, 1909	67	26 000 000	3.25%
118	"	Nov. 7, 1896	Dec. 19, 1903	50	23 100 000	3.00%
89.5	"	June, 1901	Mar. 30, 1909	88	17 900 000	3.50%
35	Railway.	Sept., 1886	1889	60	.....	.....
88	"	.....	.....	200	(Disaster, Aug. 29th, 1907.)	
50	Highway.	June, 1886	June, 1894	33.5	.....	.....
93	Railway.	July, 1912	.....	108	.....	1.20%
31	Highway.	1861	Dec., 1864	31	$360 000	.....
80	"	July, 1886	Feb., 1889	14.5	2 851 684	.....
21	Aqueduct.	.....	1842	14.5	.....	.....
29.5	Highway.	1819	1826	30	.....	Level.
2-14.5	Railway.	Apr. 13, 1846	1850	75	2 231 000	"
16.83	"	.....	May, 1859	60	.....	.....
.....	Highway.	.....	1911	.....	.....	.....
.....	Highway and railway.	.....	.....	.....	.....	.....

capacity to suit the needs of the traffic, or several bridges located so as to suit the convenience of the public, economical considerations necessitate the construction of a single large bridge, either of greater capacity than required, or which concentrates the traffic along a single path that would be better served by several.

One other point should be given due consideration, and that is the relative ease of destruction of a bridge, as compared with a tunnel. The severing of a single member would cause the collapse of a whole span. Although injury at a single spot might cause the flooding of a tunnel, the damage would be local and could be readily repaired. This is mentioned in view of the fact that national wars have not altogether ceased, and that those who disapprove of existing conditions sometimes express their dissatisfaction by the use of explosives.

Table 2 shows the general dimensions of some of the world's principal bridges crossing waters frequented by seagoing vessels. This table does not purport to be exhaustive, but merely gives typical examples of the most important bridges of the various classes, some being included merely on account of their historical significance. The physical conditions are such at certain of the bridges in Table 2

that a tunnel could not readily be substituted; at the Forth Bridge, on account of the great depth of water, which is more than 200 ft.; at the connecting railroad bridge at Hell Gate, for the same reason; at the Poughkeepsie Bridge, Washington Bridge, High Bridge, etc., on account of the height of the river banks. On the other hand, at many of the other bridge locations, in view of the modern state of the art, tunnels could be constructed more advantageously than bridges.*

Highway Tunnels: History.

London.—It is a curious fact that one of the first subaqueous tunnels, and the first on which a shield was used, was constructed as a highway tunnel, but was never used for that purpose. This was the first Thames Tunnel, built by the inventor of the shield, the late Sir Marc Isambard Brunel. His patent for a shield was No. 4 204, of 1818. Owing to London Bridge marking the head of navigation for seagoing vessels, the development of the docks and tributary industries was necessarily below this point, so that the need of some means of crossing the river below London Bridge is evident. This is indicated in Fig. 2, which shows the position of the Tower Bridge and the various tunnels below London Bridge. The Thames Tunnel, Fig. 3, crosses the river between Rotherhithe and Wapping, about 1½ miles below London Bridge. Work was started in 1825 and, after being suspended several times owing to physical and financial difficulties, was completed in 1843. In section it is one of the widest openings ever constructed, the total width of excavation being 37 ft. 6 in., and in height 22 ft. 3 in., separated by a wall 4 ft. thick, pierced by 64 arched openings, each of 4 ft. span. The length is 1 200 ft. It was proposed to have spiral approaches, but this was never done; the construction shafts, however, were fitted with stairways. In 1866, the tunnel was sold to the East London Railway Company, and since that time this company has operated its trains through it. Its cost, including shafts, was £1 300 per lin. yd.

As the Thames Tunnel had been diverted from its intended use, the vehicular traffic still had no facilities below London Bridge, but its needs were partly met by the Tower Bridge, about ½ mile below

* "Bridges and Ferry Bridges; Tunnels under Waterways used for Ocean Navigation—Economic and Technical Study," a report by Baurat Wendemuth, International Congress of Navigation, 1912.

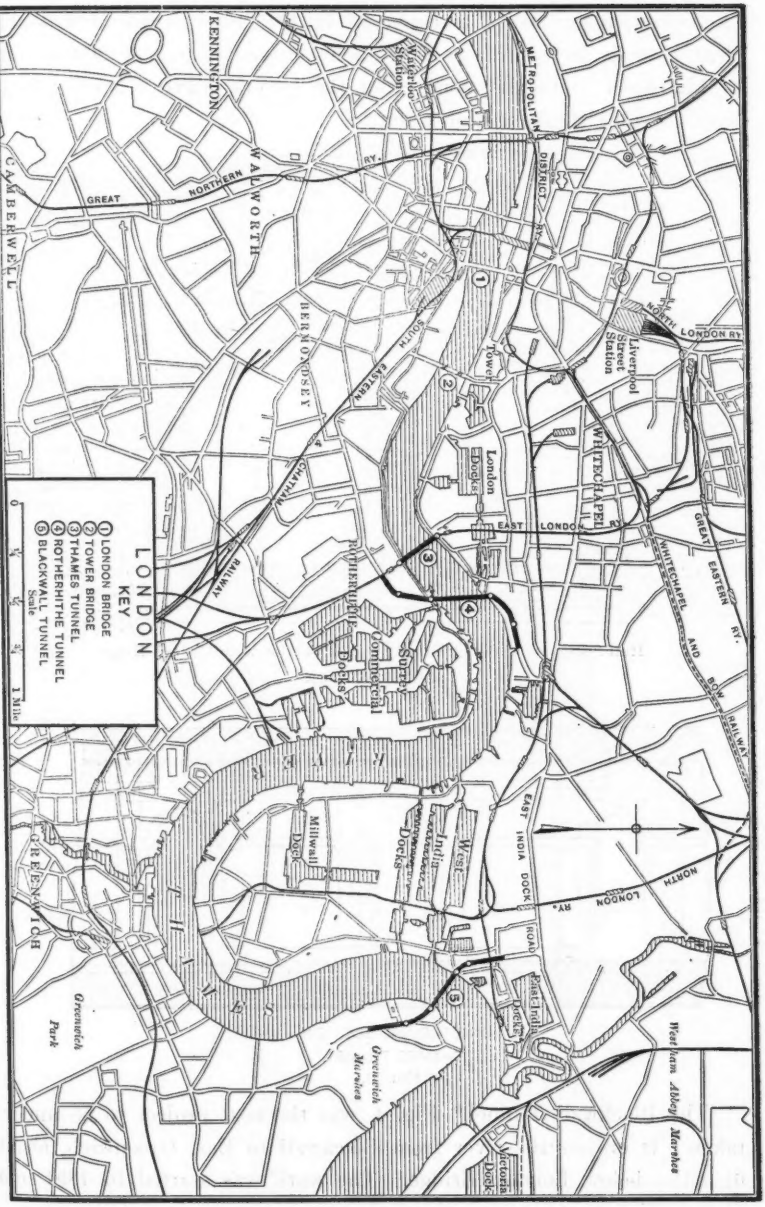
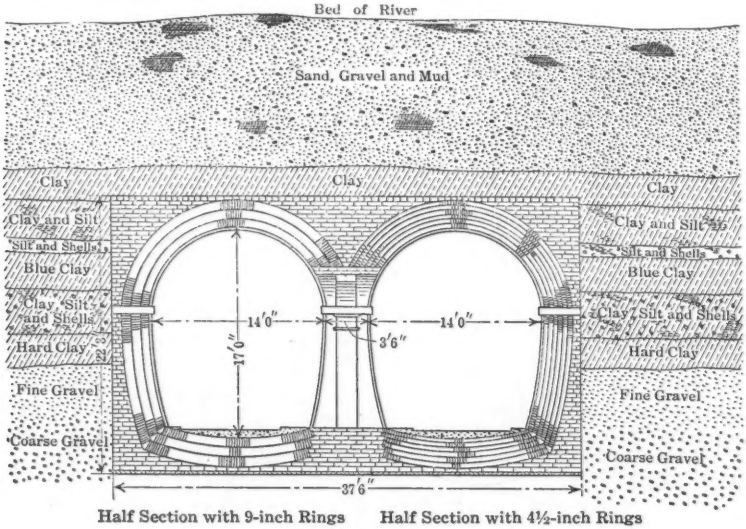
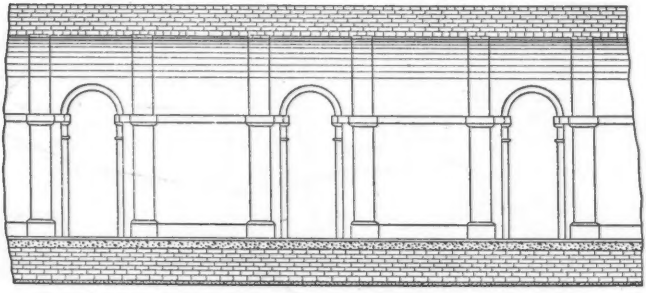

LONDON
KEY
1 LONDON BRIDGE
2 TOWER BRIDGE
3 THAMES TUNNEL
4 ROTHERHITHE TUNNEL
5 BLACKWALL TUNNEL
Scale
1 Mile
KENNINGTON
WALWORTH
CAMBERWELL
BERMONDSEY
GREENWICH
Waterloo Station
METROPOLITAN
DISTRICT
GREAT NORTHERN RY.
SOUTH EASTERN RY.
CHATHAM RAILWAY
Tower
London Docks
Liverpool Street Station
NORTH LONDON RY.
WHITECHAPEL
EAST LONDON RY.
GREAT EASTERN RY.
WHITECHAPEL AND BOW RAILWAY
Rotherhithe Docks
Commercial Docks
Surrey
THAMES
Millwall Dock
India West Docks
EAST INDIA DOCK ROAD
NORTH LONDON RY.
East India Docks
Victoria Docks
Greenwich Park
Greenwich Marshes
Western Abbey Marshes

Fig. 2.

London Bridge, opened to traffic in 1894. This consists of a double-leaf bascule draw, with a clear span between towers of 200 ft., and a high-level footbridge connecting the towers, leaving a clear head-room of 139½ ft. above high water.



CROSS-SECTION



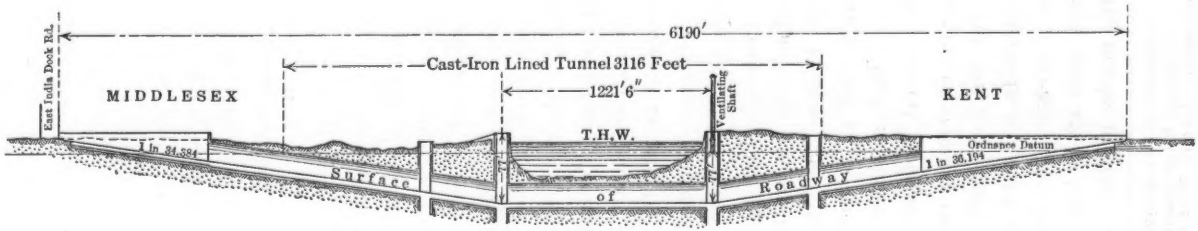
LONGITUDINAL SECTION

THAMES TUNNEL

FIG. 3.

The Blackwall Tunnel,* Fig. 4, was the next project to be undertaken. It crosses the river from Blackwall to East Greenwich, about 6¼ miles below London Bridge. The work was started in 1891 and

* *Minutes of Proceedings*, Inst. C. E., Vol. CXXX, p. 50.



PROFILE, BLACKWALL TUNNEL

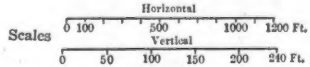
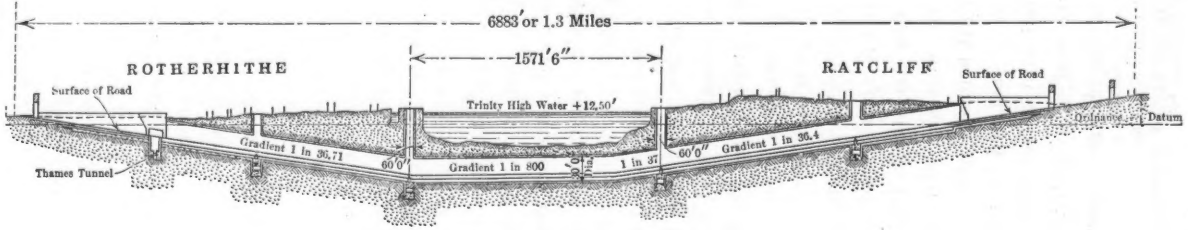


FIG. 4.



PROFILE, ROTHERHITHE TUNNEL

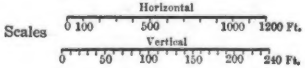


FIG. 5.

the tunnel was opened to traffic in 1897. It consists of a shield-driven, iron-lined tunnel (Fig. 6), 27 ft. outside diameter, with an internal diameter of 24 ft. 3 in. It is lined with concrete faced with glazed tile, has a roadway 16 ft. wide, and two foot-paths, each 3 ft. 1½ in. wide. The total length, including approaches, is 6 200 ft.; the length from portal to portal is 4 465 ft.; and the length under the waterway is 1 220 ft. The maximum gradient of approach is 1 in 36. Changes in direction were not made by driving the tunnel in curves, but by angles at the shafts. The cost was £871 000, amounting to about $685 per lin. ft.

The Rotherhithe Tunnel,* Fig. 5, was commenced in 1904 and completed in 1908. It crosses the river diagonally from Rotherhithe to Ratcliff, about 2¼ miles below London Bridge. The work was very similar to that at the Blackwall Tunnel. It is a shield-driven, iron-lined tunnel, with an external diameter of 30 ft. and an inside diameter (inside the concrete and tile lining) of 27 ft. It is provided with a roadway 16 ft. wide, and two walks, each 4 ft. 8½ in. wide. Figs. 8 and 9 show the Stepney Approach, and the spiral stairs at Shaft 2, Rotherhithe. The total length, including approaches, is 6 883 ft.; the length from portal to portal is 4 930 ft.; and the length under the waterway is 1 480 ft. The cost was £1 088 484, or about $930 per lin. ft.

Chicago.—In the United States the first subaqueous highway tunnel was the Washington Street Tunnel, crossing the Chicago River, in Chicago.† (Fig. 10.) This is not properly a tunnel at all, but a subaqueous passage built in an open trench between coffer-dams. It was built by the City of Chicago in 1866 to 1869, and had two roadways, each 11 ft. wide and 13 ft. high, and a footway 10 ft. wide and 10 ft. high, as shown by Fig. 11. It was at first used quite extensively for general highway traffic, and, after the fire in 1871, was the only means of communication between the west side and the business district, for some time, until the burned bridges could be restored. Incidentally, a tunnel is less likely to be damaged by fire or flood than is a bridge. This tunnel, later, fell into a bad state of repair and was used much less frequently, the traffic being largely diverted to the nearby bridges. At the time of the change of power from horse

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXV, p. 190.

† "Chicago River Tunnels: Their History and Method of Construction," by William Artingstall, *Journal*, West. Soc. of Engrs., Vol. XVI, p. 869.

1½-Inch Castings 2-Inch Castings

CROSS-SECTION, BLACKWALL TUNNEL.

FIG. 6.

CROSS-SECTION, ROTHERHITHE TUNNEL.

FIG. 7.

to cable on the Chicago street railways, permission was obtained to utilize the tunnel, one of the conditions being that the roof be lowered so as to give a depth of 17 ft. of water in the river, instead of 14 ft. This use of the tunnel was desirable, as the difficulties of operating a cable railway over a movable bridge were great. Since the substitution of electric traction for the cables, the tunnels are not so necessary, but the advantages of uninterrupted service still remain. The tunnel roof was lowered in 1889, the cost being about $135 000. This work was done by enclosing the tunnel between coffer-dams—only one-half of the river being obstructed at one time. The masonry arched roof was removed and replaced with 20-in. steel I-beams, 36 in. apart, with jack-arches between. At the same time, the dividing wall between the two roadways was removed and the tunnel was given a single opening of 20 ft. clear span. The invert was also cut out and lowered 2 ft. 6 in. in order to provide the necessary headroom. The use of the tunnel by vehicles and pedestrians was discontinued; a draw-bridge was constructed nearby; and the footway was used as a pipe gallery. In 1906 the tunnel was again reconstructed, in order to provide a navigable depth of 26 ft. instead of 17 ft. This was accomplished by cutting chases in the walls and inserting 33-in. plate girders, 48 in. apart, supporting a new masonry roof, after which the old roof was removed by breaking it up, without blasting, and dredging from the exterior. The side-walls were then underpinned and a new invert was constructed, as shown by Fig. 12. In 1909 further alterations were made in order to enable the tunnel eventually to form part of a subway system in the business district, and this necessitated building temporary inclined approaches to enable the surface cars to continue the use of the tunnel, pending the construction of the subway system. The cost of this work was about $675 000.

La Salle Street Tunnel (Chicago).—The construction of the original La Salle Street Tunnel was commenced by the City of Chicago in 1869, and it was opened for traffic on July 4th, 1871. The type of construction was similar to that of the Washington Street Tunnel. The total length is 1 887 ft., and the gradients of approach are 1 in 20. In section, it consisted of two roadways, each 11 ft. wide, and a footway 11 ft. wide and 10 ft. high, as shown by Fig. 13. The original cost was $566 000. The tunnel was given over to cable-operated street cars in 1889.

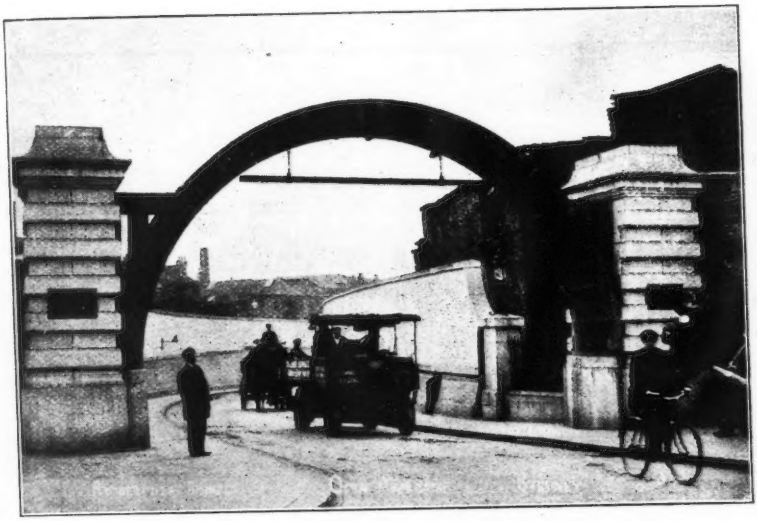


FIG. 8.—ROTHERHITHE TUNNEL: OPEN APPROACH, STEPNEY.

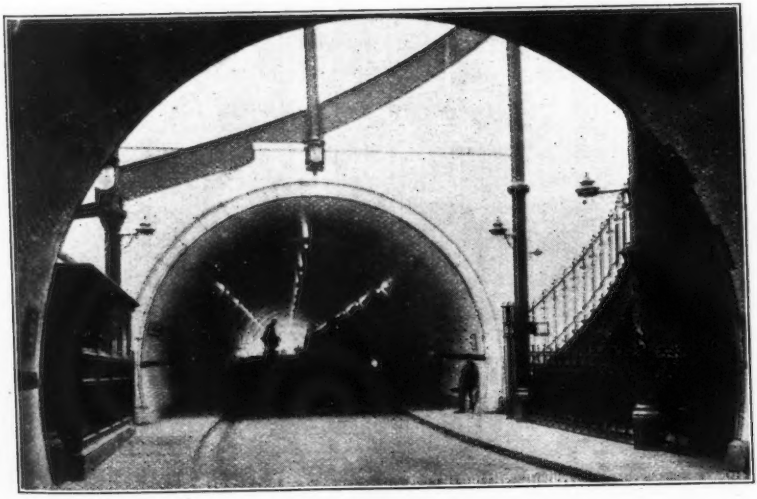


FIG. 9.—ROTHERHITHE TUNNEL: SPIRAL STAIRS, SHAFT 2.

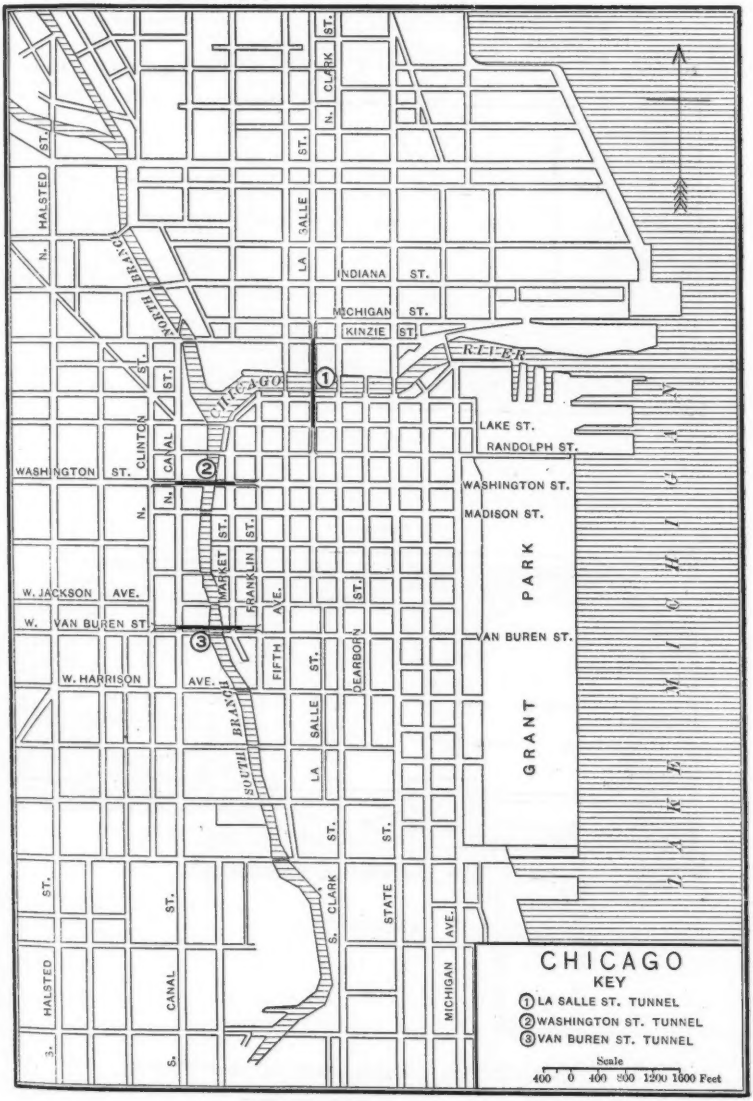

N. HALSTED ST.
N. CLARK ST.
LA SALLE ST.
INDIANA ST.
MICHIGAN ST.
KINZIE ST.
RIVER
NORTH BRANCH
CHICAGO
CLINTON ST.
CANAL ST.
WASHINGTON ST.
LAKE ST.
RANDOLPH ST.
WASHINGTON ST.
MADISON ST.
MARKET ST.
FRANKLIN ST.
W. JACKSON AVE.
W. VAN BUREN ST.
VAN BUREN ST.
W. HARRISON AVE.
FIFTH AVE.
DEARBORN ST.
GRANT PARK
SOUTH BRANCH
LA SALLE ST.
S. CLARK ST.
STATE ST.
MICHIGAN AVE.
S. HALSTED ST.
S. CANAL ST.
LAKE MICHIGAN
CHICAGO
KEY
1 LA SALLE ST. TUNNEL
2 WASHINGTON ST. TUNNEL
3 VAN BUREN ST. TUNNEL
Scale
400 0 400 800 1200 1600 Feet

FIG. 10.

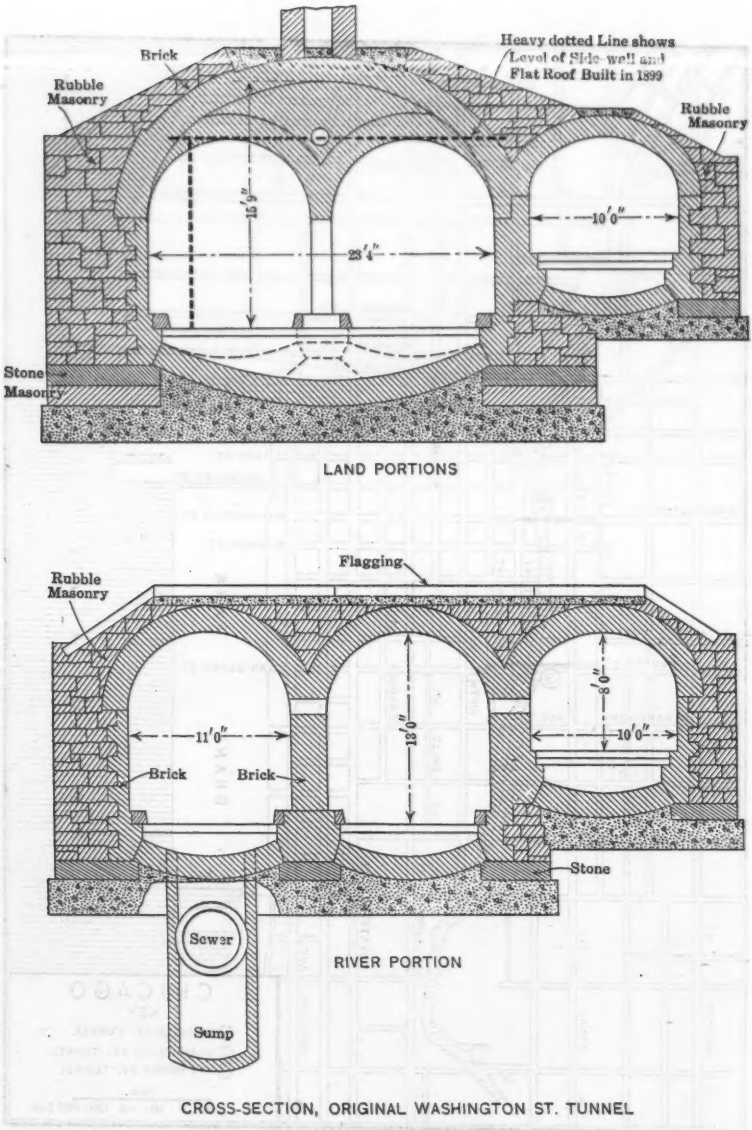
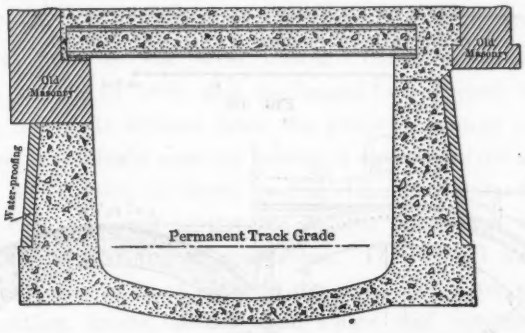

Heavy dotted Line shows
Level of Side wall and
Flat Roof Built in 1899
Brick
Rubble
Masonry
Rubble
Masonry
15′0″
10′0″
23′4″
Stone
Masonry
LAND PORTIONS
Flagging
Rubble
Masonry
8′0″
11′0″
13′0″
10′0″
Brick
Brick
Stone
Sewer
Sump
RIVER PORTION
CROSS-SECTION, ORIGINAL WASHINGTON ST. TUNNEL
0
5
10
15 Feet

FIG. 11.

In 1906 work on the lowering of the roof, in order to increase the navigable depth of water from 18 to 26 ft., was commenced. It was intended to follow a method similar to that used for the Washington Street Tunnel. Inside the tunnel, 30-in. steel girders were to rest on the old west wall, and a new brick wall was to be placed inside the old footway, the latter being retained as a pipe gallery. Owing to the development of excessive leaks in executing this work, the tunnel was flooded and a change of plan was made. A method of doing the work by enclosing sections in coffer-dams was considered, but, as the United States Government engineers would not permit coffer-dams for a distance of 60 ft. from the dock lines, it was necessary to devise some other plan. The method adopted was to construct



CROSS-SECTION, FINAL WASHINGTON ST. TUNNEL.

0 5 10 15 Feet

FIG. 12.

two parallel steel cylinders, each comprising about 75% of a circle, with a common longitudinal stiffening truss. This shell was built at a dry dock, and was 278 ft. long, 27 ft. high, and 41 ft. wide. It was made up of $\frac{3}{8}$-in. plates, 7 ft. 6 in. wide, and 19 ft. long, stiffened by Z-bars and plate and angle gussets. The concrete for the invert and center wall was placed while the shell was in the dry dock, as were the timber bulkheads in each end, and the temporary internal bracing, as shown by Fig. 14. The weight of the steel in the tubes was about 500 tons, and the total weight when first floated was about 3 000 tons. After floating, the shell was towed to near the site of the tunnel. Internal concrete was then added until the tops of the tubes barely projected above the water, the total displacement being 7 852

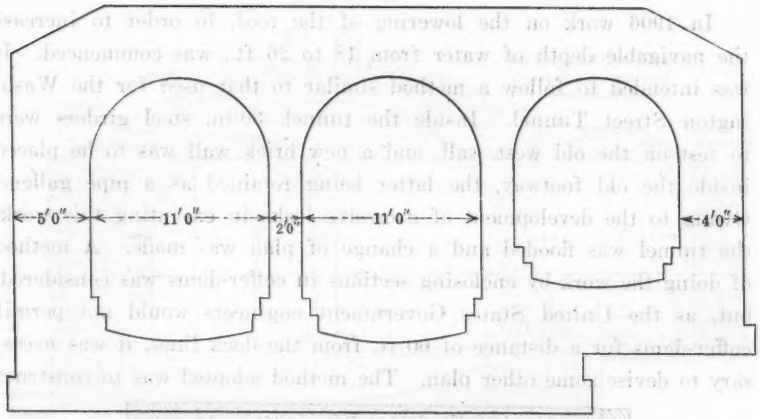

5′0″
11′0″
2′0″
11′0″
4′0″
CROSS-SECTION, ORIGINAL LA SALLE ST. TUNNEL
0 5 10 15 Feet

FIG. 13.

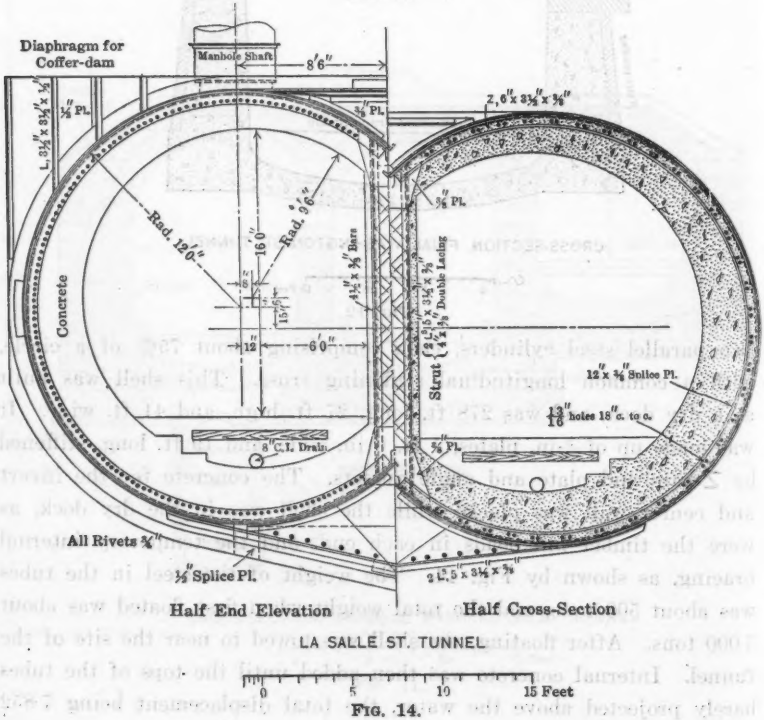
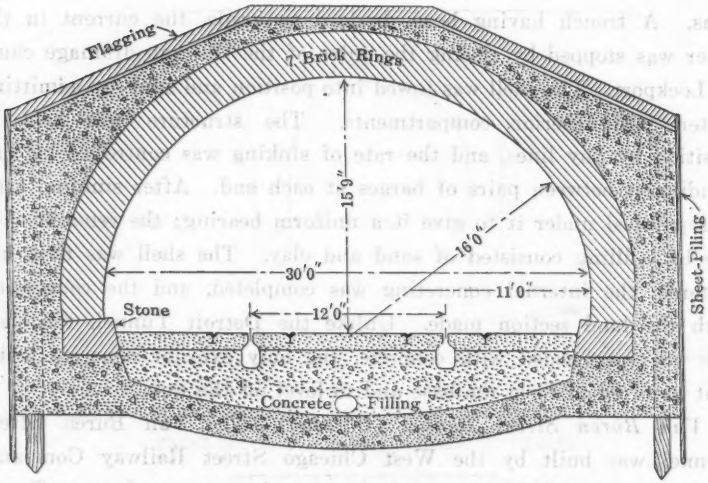

Diaphragm for Coffer-dam
Manhole Shaft
8′6″
L 3½″ x 3½″ x ½″
½″ Pl.
¾″ Pl.
7,6″ x 3½″ x ⅜″
⅝″ Pl.
Rad. 12′0″
Rad. 9′6″
16′0″
Concrete
12″
6′0″
15″
1½″ x ¾″ Bars
Strut { 2 L 6″ x 3½″ x ⅜″ 1 4″ x ¾″ Double Lacing
12″ x ⅝″ Splice Pl.
13″ 18 holes 18″ c. to c.
8″ C.I. Drain
⅝″ Pl.
All Rivets ¾″
½″ Splice Pl.
2 L 5″ x 3½″ x ⅝″
Half End Elevation
Half Cross-Section
LA SALLE ST. TUNNEL
0 5 10 15 Feet

FIG. 14.

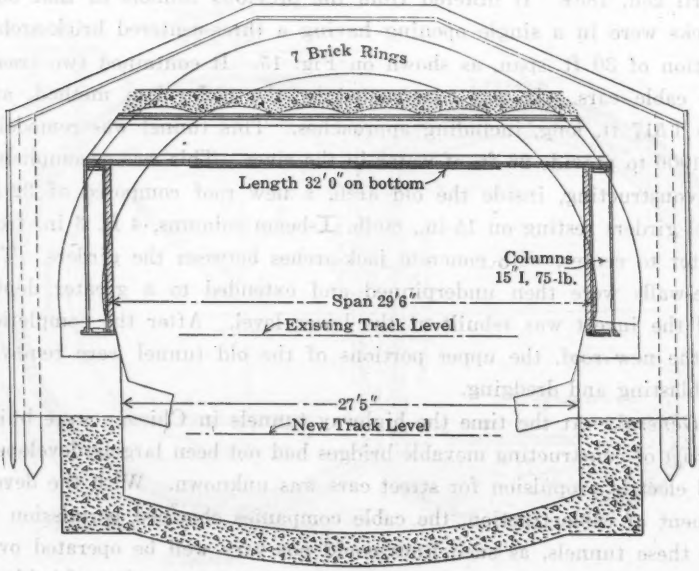
tons. A trench having been dredged to grade, the current in the river was stopped by closing the gates of the Chicago drainage canal at Lockport. The shell was towed into position and sunk by admitting water into interior compartments. The structure was held in position by guy lines, and the rate of sinking was controlled by suspending it between pairs of barges at each end. After sinking, sand was pumped under it to give it a uniform bearing; the remainder of the back-filling consisted of sand and clay. The shell was then unwatered, the internal concreting was completed, and the connection with the land section made. Unlike the Detroit Tunnel, the shell was not surrounded with concrete, the only exterior concrete being that required to form the base and to level off the top.

Van Buren Street Tunnel (Chicago).—The Van Buren Street Tunnel was built by the West Chicago Street Railway Company. Work was started in 1889, and the tunnel was opened to traffic on April 23d, 1894. It differed from the previous tunnels in that both tracks were in a single opening having a three-centered brick-arched section of 30 ft. span, as shown on Fig. 15. It contained two tracks for cable cars. It was constructed by the coffer-dam method, and was 1 517 ft. long, including approaches. This tunnel was remodeled in 1906 to provide 26 ft. of water in the river. This was accomplished by constructing, inside the old arch, a new roof composed of 32-in. steel girders resting on 15-in., 80-lb. I-beam columns, 4 ft. 3 in. from center to center, with concrete jack-arches between the girders. The side-walls were then underpinned and extended to a greater depth, and the invert was rebuilt at the lower level. After the completion of the new roof, the upper portions of the old tunnel were removed by blasting and dredging.

General.—At the time the highway tunnels in Chicago were built, the art of constructing movable bridges had not been largely developed, and electric propulsion for street cars was unknown. With the development of cable traction, the cable companies obtained permission to use these tunnels, as such lines could not very well be operated over movable bridges. At the same time, the increasing number of bridges, together with improvements in the mechanism for operating them—which reduced the delays due to the draw being open—and the avoidance of the heavy gradients of the tunnel approaches, led to their



CROSS-SECTION, ORIGINAL VAN BUREN ST. TUNNEL



CROSS-SECTION, FINAL VAN BUREN ST. TUNNEL

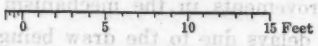


FIG. 15.

diminishing use by vehicular traffic, and finally to their being turned over exclusively to the street-car lines.

The original Washington Street Tunnel was in very leaky condition from the beginning, and in the winter great trouble was caused by the formation of ice—so much so, that in severe weather the accumulation of ice and icicles could not be removed fast enough, and the tunnel had to be closed to traffic.

Glasgow.—In Glasgow, a tunnel for vehicles was constructed across the Clyde in 1890-93. Access to this tunnel, instead of being obtained by inclined approaches, is by elevators in shafts. The tunnel consists of three separate cast-iron tubes, each having an internal diameter of 16 ft., the center tube being used as a footway and the outer ones for vehicles, each in a single direction. Fig. 16 shows the profile. The footway tube has inclined approaches and stairs. The shafts are circular, 76 ft. inside diameter, and 80 ft. outside diameter. The river is 415 ft. wide, and the length of the tunnel from shaft to shaft is 700 ft. This tunnel was constructed by private capital, but, owing to competition by a municipal ferry, its use has been discontinued. It is now, however, proposed to re-open it, the City Corporation having arranged with the Glasgow Harbor Tunnel Company to operate it for a period of one year after the date of re-opening, on condition that, on the expiration of that time, the Corporation shall have the option of purchasing the whole undertaking for £100 000, although the original cost was £287 000.*

Hamburg.—A highway tunnel, Fig. 17, very similar to that at Glasgow, has just been completed under the Elbe at Hamburg.† Access to this tunnel is also obtained by elevators in shafts, and the total length from center to center of shafts is 1 471 ft. Each shaft (72 ft. inside diameter and 84 ft. outside), contains four elevators for vehicles and two for foot passengers. The lift of the elevators is 78 ft. The lining is composed of structural-steel plates and angles, the outer diameter being 19.7 ft. It has a lining of concrete and tile, and the roadway is only wide enough for a single vehicle, there being a separate tube for traffic in each direction. Fig. 18 is a cross-section. The general dimensions of the subaqueous highway tunnels of the world are given in Table 3.

* *Engineering*, May 10th and 31st, June 14th and 28th, 1895, contains a description of the shafts, lifts, etc.

† *Zeitschrift*, Verein Deutscher Ingenieure, August 17th, 1912.

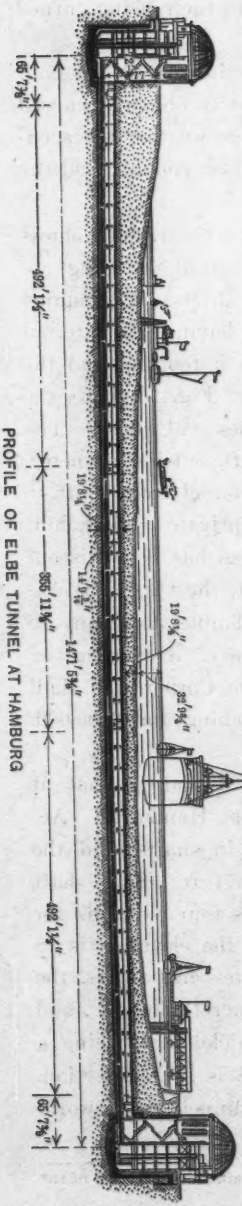
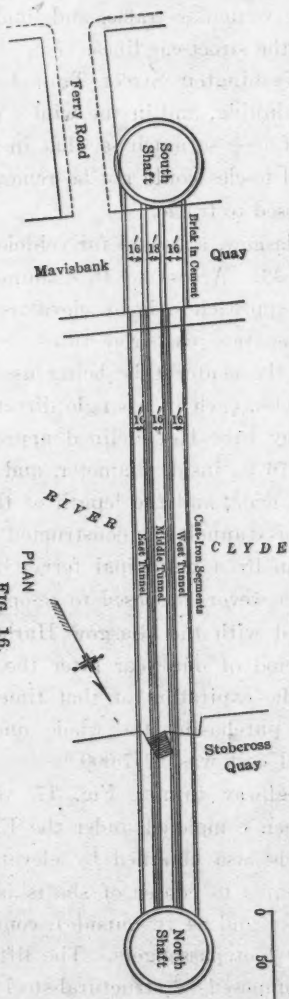
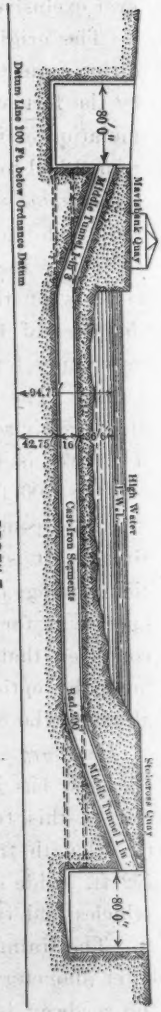


FIG. 17.



PLAN

FIG. 16.



PROFILE

GLASGOW HARBOR TUNNELS

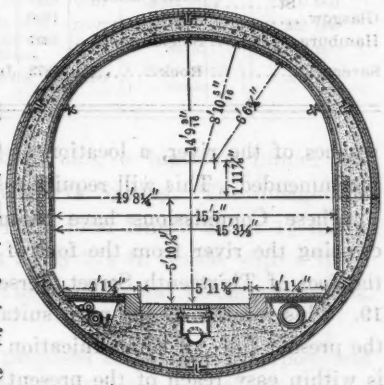
0 50 100 150 200 250 Feet

Projects.

From the preceding it will be seen that highway tunnels have not had a very fortunate history. The Thames Tunnel, the first ever built, was never utilized for the purpose intended, but was first used by a steam railway, and now by a railway operated by electric power. The tunnel, however, is now performing a useful function, and is practically in as good condition as when completed, 70 years ago, carrying more vehicles, each of much greater weight, than was contemplated originally, and it is not probable that as much could be said of a bridge, other than one of masonry.

The Chicago tunnels have had to be rebuilt to provide greater depth of water (one of them being rebuilt twice), and their use for general highway traffic has been discontinued. They are now being utilized only by the street cars.

The use of the Glasgow Tunnel has been discontinued, owing to municipal competition, and only the Blackwall, Rotherhithe, and Hamburg Tunnels remain in successful operation. Nevertheless, various projects are under consideration for the construction of highway tunnels, and therefore the subject is worthy of consideration.



CROSS-SECTION, OF ELBE TUNNEL AT HAMBURG

Fig. 18.

New York.—Some means other than ferries for vehicles to cross the Hudson (North) River at New York has long been needed. As the river forms the boundary between the States of New York and New Jersey, such means of crossing must be provided by the joint action of the two States or by a private corporation. This subject is now under investigation by the two States, through the medium of the New Jersey Interstate Bridge and Tunnel Commission and the New York Interstate Bridge and Tunnel Commission. These two Commissions have agreed that, if a bridge is to be built, it should be as far south as physical conditions will permit. As no suitable foundation can be obtained at points where piers would be permitted in the lower

TABLE 3.—GENERAL DIMENSIONS OF THE

Location.	Method of construction.	Dates.		Length under waterway, in feet.	Length, portal to portal, in feet.	Total length including approaches, in feet.	Depth to invert, in feet.
		Work begun.	Work finished.				
Thames	Shield	1825	1843	925	1 200	.....	65
Blackwall	Shield	1891	1897	1 220	4 465	6 200	80
Rotherhithe	Shield	1904	1908	1 460	4 930	6 883	75
Chicago, Washington St.	Coffer-dam	1866	1869	152	934	1 608	36
Chicago, Washington St.	Reconstructed	1906	1910	152	.....	1 550	53
Chicago, La Salle St.	Coffer-dam	1869	1871	285	1 505	1 890	38
" " "	Reconstructed	1906	....	285	.....	2 030	50
" Van Buren St.	Coffer-dam	1889	1894	184	920	1 514	44
Glasgow	Shield	1890	1898	415	700	.....	65
Hamburg	Shield	1907	1911	1 214	1 471	.....	72
Severn	Rock	1873	July 1, 1887	2¼ miles, 1 650 ft.	22 998	6½ miles	156

reaches of the river, a location at about Fifty-eighth Street has been recommended. This will require a span of 2 880 ft.

These Commissions have recommended a location for a tunnel crossing the river from the foot of Canal Street, New York, to about the foot of Thirteenth Street, Jersey City, as shown on the map, Fig. 19. This location is the most suitable, as it is in the average path of the present lines of communication for vehicles crossing the river, and is within easy reach of the present bridges over the East River. The New York approach to such a tunnel will come to the surface at about the intersection of Varick and Canal Streets. Varick Street is now being widened, and will connect with the extension of Seventh Avenue, which is now under way.

The present traffic crossing the river at New York is about 20 000 vehicles daily.

Boston.—In Boston there has been some agitation for a highway tunnel from Boston to East Boston. Gen. J. G. Foster, when United States Engineer Officer at that port, in 1868, prepared a design for such a tunnel.

Chicago.—In Chicago it has been proposed to build a tunnel under the Chicago River in order to connect the Boulevard and Park System. Two tunnels, each 31 ft. in diameter, have been suggested.

WORLD'S SUBAQUEOUS HIGHWAY TUNNELS.

Depth of water, in feet.	No. of tunnels.	Size of Tunnels.		Kind of lining.	Rate of gradient of approach.	Character of strata tunneled through.	Cost.
		Internal.	External.				
32	Twin	16′ 4″ × 13′ 9″	22′ 3″ × 37′ 6″	Brick	.....	Clay & gravel	$2 110 per ft.
40	1	24′ 3″ dia.	27′ 0″ dia.	Cast iron	1:36	Cl., sand & gr.	4 242 000
40	1	27′ 0″ dia.	30′ 0″ dia.	Cast iron	1:36.5	" " " "	5 300 000
14	3	2–13′ 0″ × 11′ 0″ 1–10′ 0″ × 10′ 0″	.....	Brick	1:16	Stiff blue clay	512 707
26	1	1–21′ 0″ × 25′ 0″	.....	Concrete	1:10	" " "	.....
17	3	.....	.....	.....	1:20	" " "	566 000
27	2	19′ 0″	24′ 0″	Concrete	1:33.3	" " "	.....
18	1	15′ 9″ × 30′ 0″	28′ 0″ × 42′ 0″	Brick & con.	1:10	Soft clay	1 800 000
49	3	16′ 0″ dia.	17′ 0″ dia.	Cast iron	.....	Sand & gr.	1 898 000
33	2	14.8′	19.7′	Structural steel.	....	Sand	2 386 000
96.30	1	25′ 0″ × 26′ 0″	30′ 0″ × 31′ 0″	Brick	1:90	Sandstone shale	7 800 000

Sydney.—At Sydney, New South Wales, a highway tunnel has been proposed to connect Sydney and North Sydney. The subaqueous portion would be 1 416 ft. long, and the total length about 1½ miles. The proposed section would be about 21 by 27½ ft. inside, with a 16-ft. roadway and two 4½-ft. walkways. The more recent recommendations, however, have been for a bridge.

Liverpool.—It has recently been proposed to connect Liverpool and Birkenhead by a tunnel under the Mersey for the use of vehicles and tram cars. As a successful passenger railway tunnel has been constructed under the river at this point, there would probably be no insurmountable physical difficulty in building a similar tunnel for highway purposes.

Oakland, Cal.—A tunnel has been proposed between Oakland and Alameda for a 20-ft. driveway and two 6-ft. suspended foot-walks.

METHODS OF CONSTRUCTION.

Existing highway tunnels have been constructed in soft ground by the shield method, by coffer-dam, or by sinking a metal form in a trench dredged from the surface, in which a lining of concrete was deposited; but, as other methods have been used successfully in build-

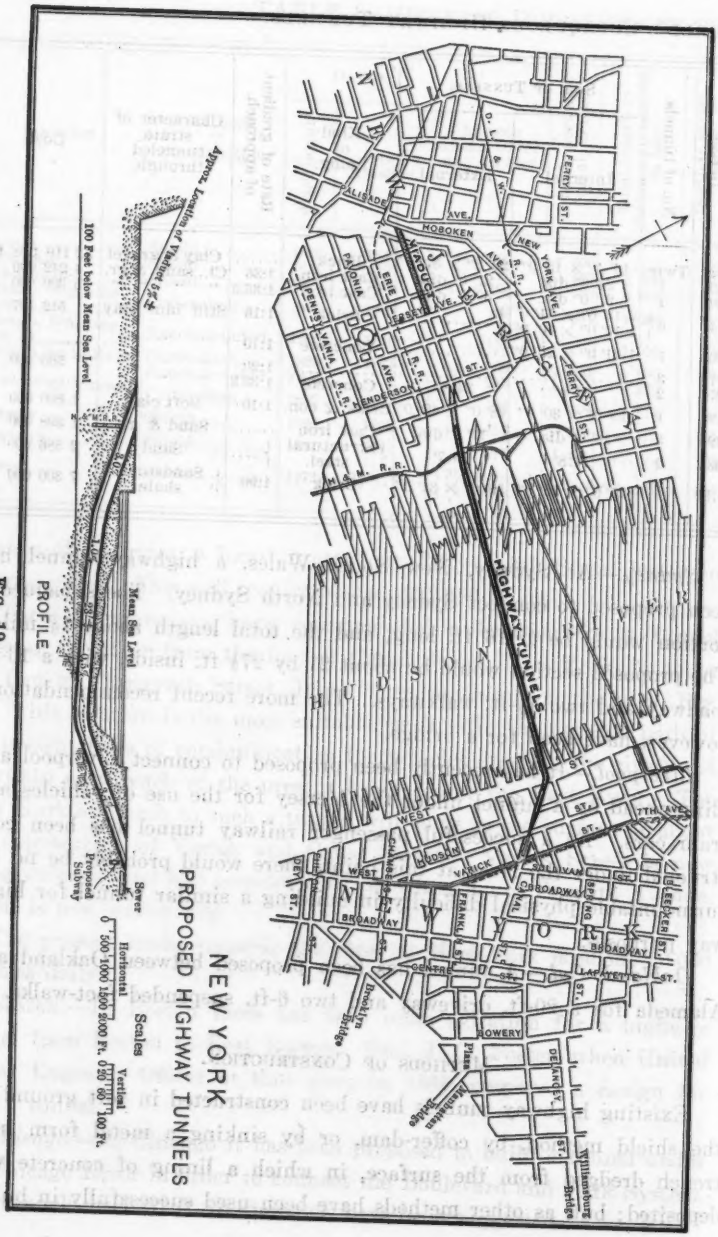

NEW YORK
PROPOSED HIGHWAY TUNNELS
HIGHWAY TUNNELS
HUDSON RIVER
NEW YORK BAY
HOBOKEN
JERSEY CITY
Brooklyn Bridge
Manhattan Bridge
Williamsburg Bridge
Scales
Horizontal
0 500 1000 1500 2000 Ft.
Vertical
0 50 60 100 Ft.
PROFILE
Mean Sea Level
100 Feet below Mean Sea Level
Proposed Subway
Sewer

FIG. 19.

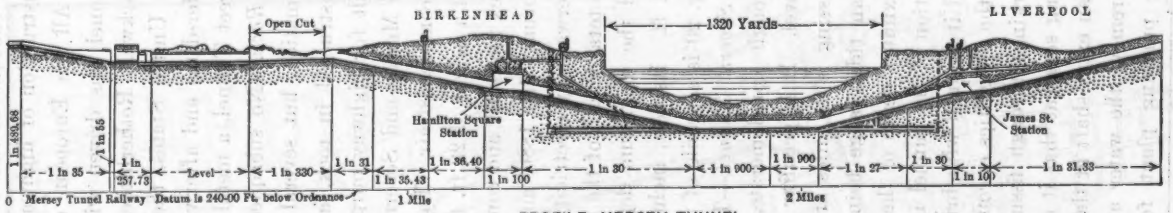
ing subaqueous tunnels for other purposes, they can be applied to the construction of highway tunnels where the conditions are favorable.

All the European examples are shield-driven. The first Thames Tunnel was driven without compressed air, but it was used on the Blackwall, Rotherhithe, Glasgow Harbor, and Hamburg Tunnels. In the United States, the Chicago tunnels were originally built within coffer-dams and afterward lowered by various means; in the La Salle Street Tunnel, a metal form was sunk from the surface.

Rock.—No subaqueous highway tunnels have been driven in rock formation, but several notable tunnels for other purposes have been constructed in rock, and there is no reason that they could not be built for highway purposes where the conditions are favorable. Both the Mersey and Severn Tunnels were driven in rock without the aid of compressed air. The Mersey Tunnel has an internal span of 26 ft. and a height of 23 ft. from the invert to the intrados. It is lined with brick masonry, and contains a double-track railway. It was opened on February 1st, 1886, and was first worked with steam locomotives and afterward by electricity. It is 3 960 ft. long between quay walls, and the total length of the line is 4.1 miles. It was excavated in sandstone, and the maximum depth of invert below mean high water is about 151 ft. Its cost, including equipment, was about £500 000 per mile.* Fig. 20 is a profile of this tunnel, and Fig. 22 is a cross-section.

Severn Tunnel.—The Severn Tunnel, Fig. 21, was excavated through conglomerate, carboniferous strata, sandstone, shale, and gravel. It is peculiar in that the width of the river at the point of crossing is 2¼ miles at high tide, but only 1 650 ft. at low water, the mean tidal range being 37 ft. Its total length is 4.36 miles. The maximum depth of the invert below high tide is 156 ft. The internal section of the tunnel is 20½ ft. high above rail level, and 26 ft. wide, and it is lined with brick masonry. The tunnel is notable on account of the difficulties encountered during construction. Water reached the workings through fissures and springs, the pressure from one spring being so great that it crushed the brickwork. This led to the sinking of an extra shaft at the site of the spring in which pumps were placed to remove the water and relieve the pressure. The total capacity of the pumping plant for the permanent drainage of the tunnel is

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXXVI, p. 40.



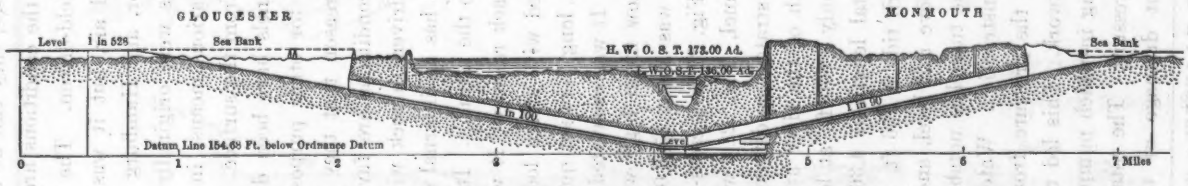
PROFILE, MERSEY TUNNEL

Scales

Horizontal
1000 750 500 250 0 1000 2000 Feet

Vertical
100 0 100 200 Feet

FIG. 20.



PROFILE, SEVERN TUNNEL

Scales

Horizontal
1 ¾ ½ ¼ 0 1 Mile

Vertical
100 50 0 100 200 Ft.

FIG. 21.

66 000 000 gal. per day, this being about twice the maximum quantity of drainage. The total cost of the tunnel was about £1 500 000.*

Various subaqueous tunnels have been constructed in rock for the conveyance of gas and water, some of these being at great depth, but the conditions of access are so different from those of a highway tunnel that they are scarcely comparable.

The conditions favorable to such tunnels in rock are where the strata are non-water-bearing, without faults or fissures, or where the depth of the water is beyond the maximum for work with air pressure. One advantage is that the tunnels can be given an economical cross-section more suited to their use than the circular form usually obtained with a shield.

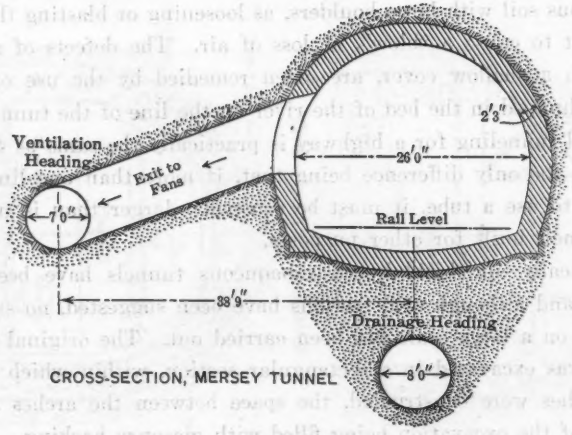


FIG. 22.

Shield Tunnels.—Four of the subaqueous highway tunnels have been driven in soft ground with a shield, and with the aid of compressed air. If the Thames Tunnel is included, there are five, but, in this latter case, compressed air was not used. Two of these tunnels (the Glasgow and the Hamburg), provide for a single line of vehicles in a tube, and two (the Blackwall and the Rotherhithe), provide two lines in opposite directions in a single tube, with a footway as well. The diameters of these tunnels range from 17 to 30 ft. A shield tunnel for a double line of electric railway has recently been completed under the Seine, in Paris, with a diameter of 25 ft. $6\frac{5}{32}$ in.

* "The Severn Tunnel," by T. A. Walker.

The conditions favoring the selection of the shield method of driving tunnels are: where the strata are of water-bearing silt, clay, sand, or gravel; where the bottom of the tunnel is not more than 100 ft. below the water surface; where a reasonable cover of material, between the top of the tunnel and the bed of the river, can be obtained; and where the channel of the waterway cannot be obstructed during construction. In suitable material very rapid progress can be made, the cast-iron lining facilitating rapid construction. On one tunnel of the Hudson and Manhattan Railroad, driven through Hudson River silt, a length of 72 ft. was built in one day.

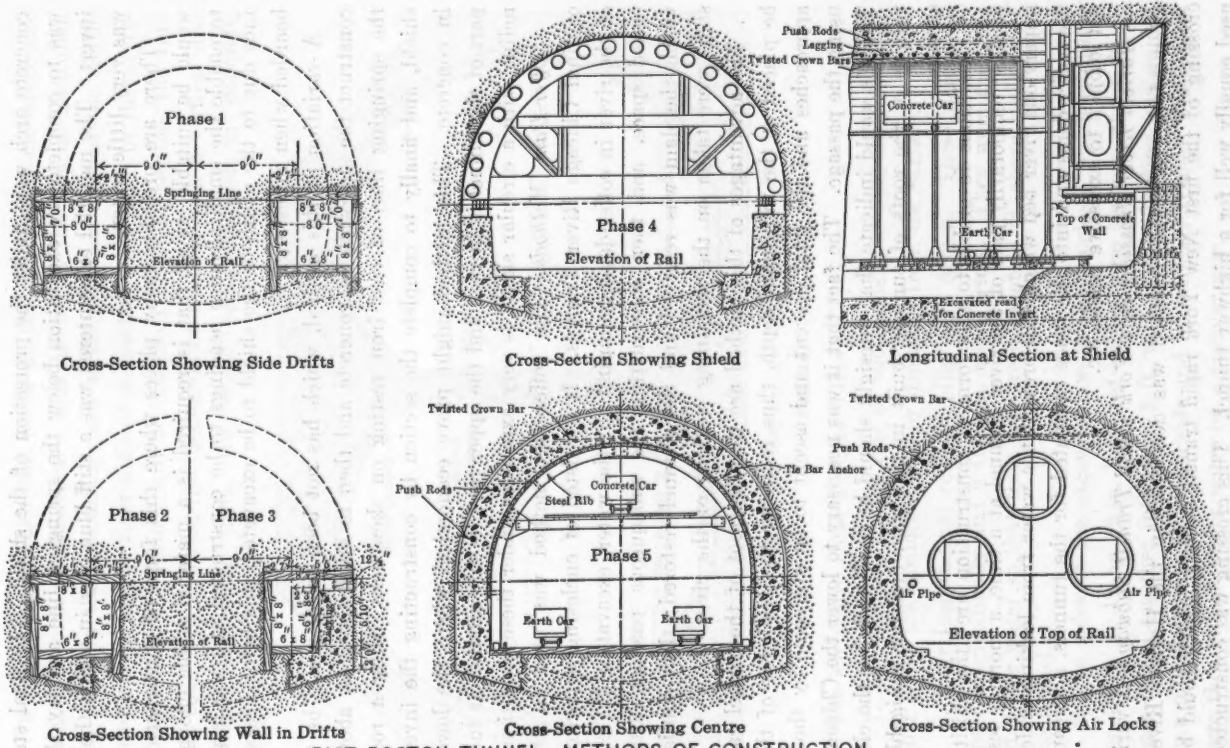
Shield construction is difficult when the lower portion of the tunnel is in rock and the upper portion is in silt or sand; it is also difficult in a porous soil with large boulders, as loosening or blasting the boulders is apt to cause a "blow", or loss of air. The defects of a porous soil, with a shallow cover, are often remedied by the use of a clay blanket dumped in the bed of the river on the line of the tunnel.

Shield tunneling for a highway is practically the same as for other purposes, the only difference being that, if more than one line of vehicles is to use a tube, it must be somewhat larger than is usual for such tunnels built for other purposes.

Practically all shield-driven subaqueous tunnels have been made circular, and although oval sections have been suggested, no such construction on a large scale has been carried out. The original Thames Tunnel was excavated in a rectangular section, within which the two brick arches were constructed, the space between the arches and the exterior of the excavation being filled with masonry backing.

Roof Shields.—No highway tunnels have been built with a roof shield, although this form of construction would give a more suitable section than a circular shield. A notable example of this form of tunneling is the East Boston Tunnel,* used by a double line of street cars. The internal span of this tunnel is 23.33 ft., with a height of 18 ft. above the rails. It was lined with concrete, which was about 3 ft. thick. The method of procedure, Fig. 23, was to excavate two small headings—one on each side—in which the sidewalls were constructed. This was followed by the shield, which rested on the sidewalls and was advanced by the jacks thrusting on push-bars embedded in the concrete. The excavation above the springing line and the

* Seventh Annual Report, Boston Transit Commission, August 15th, 1901.



EAST BOSTON TUNNEL. METHODS OF CONSTRUCTION.

FIG. 23.

concrete arch were under the protection of the shield. The final step was to complete the excavation below the springing line and lay the invert. The material encountered was a stiff blue clay in which there was very little water.

There are probably many places where this form of construction would be suitable, as all that is required is a material stable enough to enable the small advance headings to be constructed and the large face due to the width of the shield to be excavated, this being breast-boarded when necessary.

A variation of this method, which has not been tried, would be to construct the side-walls of concrete and then to build the roof, above the springing line, of cast iron resting on skewbacks, using a roof shield, and finally to complete the section by constructing the invert in concrete. This method might prove economical where the lower part of the tunnel is in rock and the upper part in soft ground, as tunneling with a circular shield is very expensive under these conditions.

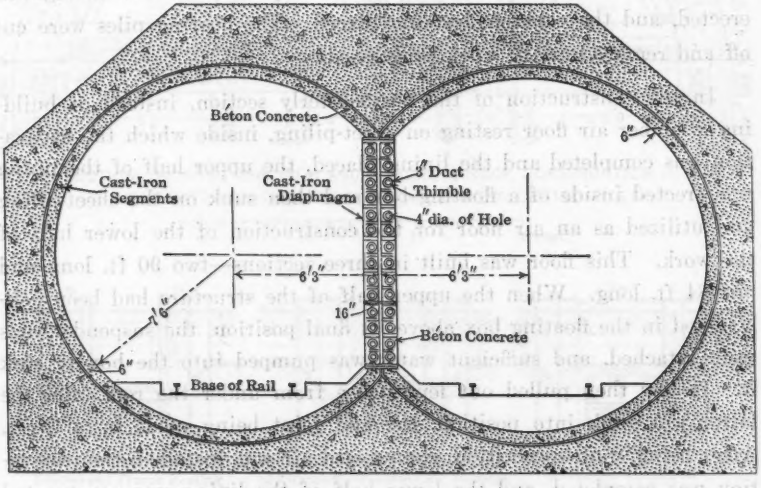
Coffer-Dam Methods.—The coffer-dam method was used in the original Chicago River Tunnels. It consisted of enclosing a portion of the river in coffer-dams, pumping out the water, excavating the enclosed space, and then constructing the subaqueous passage, after which the dam was removed and the channel restored; then similar steps were taken on the remaining sections of the river.

The advantages of this method are that the roof of the tunnel can be placed at the minimum depth, thus reducing the gradients of the approaches and the total descent and ascent to be overcome by those using the passage. The fact that it was necessary to lower the Chicago Tunnels would indicate that foresight should be used in fixing the elevations of the roofs of tunnels, built in this way, crossing navigable streams.

Conditions favorable to this form of construction are: the ability to close temporarily part of the river channel at a time, a more or less impervious river bed which will prevent the water from leaking under the coffer-dams and into the space excavated for the tunnels, a stream not subject to excessive floods, etc.

Harlem River Crossing, New York Rapid Transit Subway.—A variation of the coffer-dam method was developed in the Harlem River crossing of the first New York rapid transit subway, which could be used equally well for a highway tunnel. This crossing was constructed

in two sections, one-half the width of the river being closed at one time. The plan of construction was devised by the firm of McMullen and McBean, the contractors, and as the methods used in the southwesterly half of the river differed somewhat from those in the northeasterly half, the former will be described first. In section, the tunnels consist of two cast-iron segments of a circle with a vertical diaphragm separating them, the whole being encased in concrete. (Fig. 24.)



CROSS-SECTION, HARLEM RIVER TUNNEL

FIG. 24.

The successive steps were as follows:

1. The material on the line of the tunnel was excavated to somewhat below the springing line by dredging from the surface.

2. Four longitudinal rows of piles were driven. These were 8 ft. apart longitudinally and 6 ft. 4 in. transversely. They were then cut off under water so that their tops were about 11 ft. above the axis of the tunnel.

3. The sides of the excavation were then enclosed by sheet-piling, consisting of 12 by 12-in. long-leaf yellow pine timbers, about 65 ft. long, bolted together into units 3 ft. wide. This sheeting was guided by a pile platform along each side and by a timber frame. These piles were then cut off under water by a circular saw.

4. On top of the four rows of bearing piles and the sheet-piles, was then sunk a timber deck consisting of three transverse layers of 12 by 12-in. timbers and two longitudinal layers of 2-in. plank. After this was sunk accurately in place, it was covered with a thickness of about 5 ft. of earth and mud.

5. Several vertical shafts were placed, passing through this deck, and, after placing bulkheads at the ends and air locks, compressed air was turned on and the excavation was completed, the iron lining was erected, and the concrete backing placed. The bearing piles were cut off and removed as the work progressed.

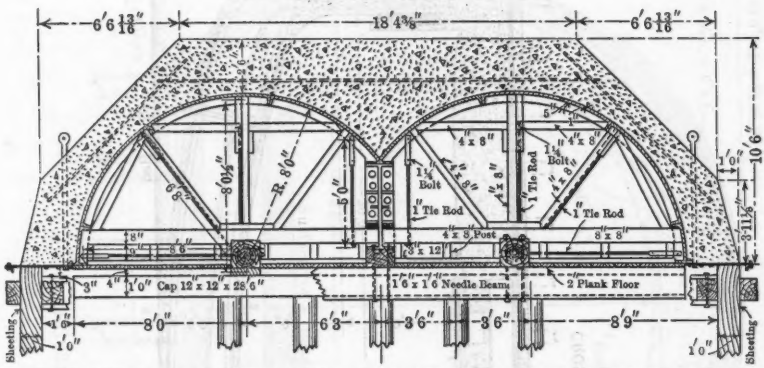
In the construction of the northeasterly section, instead of building a timber air floor resting on sheet-piling, inside which the excavation was completed and the lining placed, the upper half of the lining was erected inside of a floating box and then sunk on the sheet-piling and utilized as an air floor for the construction of the lower half of the work. This floor was built in three sections—two 90 ft. long and one 84 ft. long. When the upper half of the structure had been constructed in the floating box above its final position, the suspender rods were attached, and sufficient water was pumped into the box to sink it. It was then pulled out lengthwise from under the roof, and the latter was sunk into position, the end joint being made by a diver. After being sunk to position, compressed air was installed, the excavation was completed, and the lower half of the lining was constructed (Fig. 25).

A design for the application of this method to a highway tunnel was prepared by the Carnegie Steel Company and presented in a paper by R. B. Woodworth, M. Am. Soc. C. E., before the Railway Club of Pittsburgh.* (Fig. 26.) This plan contemplates a roadway 60 ft. wide, with two electric car tracks, the floor and side-walks being of concrete and the roof of steel embedded in concrete.

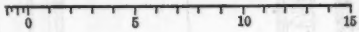
Another design for a highway tunnel by this system, having two roadways, each 19 ft. 6 in. in width, was shown before the Hoboken Board of Trade on February 4th, 1913. (Fig. 27.)

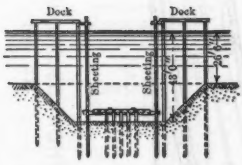
Detroit Tunnel.—The method used in constructing the Detroit River Tunnel has been described so thoroughly in the paper† by W. S.

* *Official Proceedings*, Railway Club of Pittsburgh, December 22d, 1909.
† *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 288.

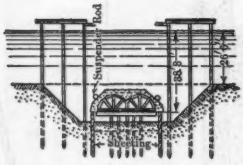


CROSS-SECTION, SHOWING THE IRON AND CONCRETE ROOF CONSTRUCTION IN PLACE SUPPORTED BY THE SIDE-SHEETING AND THE CENTER PILES.

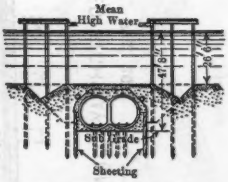




Tunnel Trench, Docks and Submerged Frame Resting on Pile Caps.

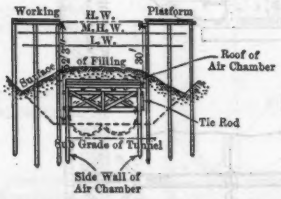


Sheeting Cut-off and Upper Part of Tunnel Shell in Position, Forming the Roof and Side-walls of a Working Chamber.



Section of Completed Tunnel.

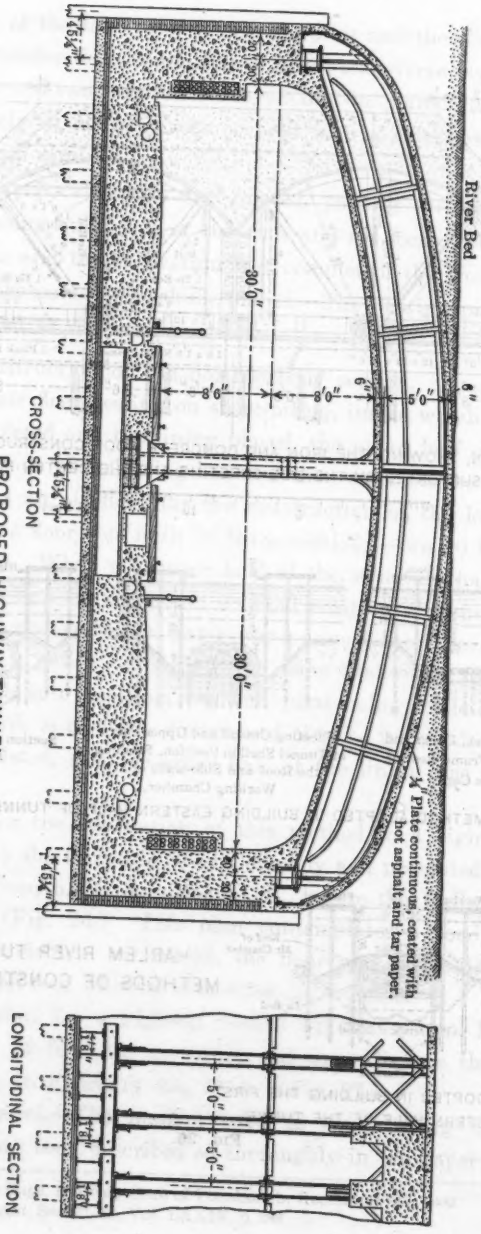
METHOD ADOPTED IN BUILDING EASTERN HALF OF TUNNEL.



HARLEM RIVER TUNNEL.
METHODS OF CONSTRUCTION.

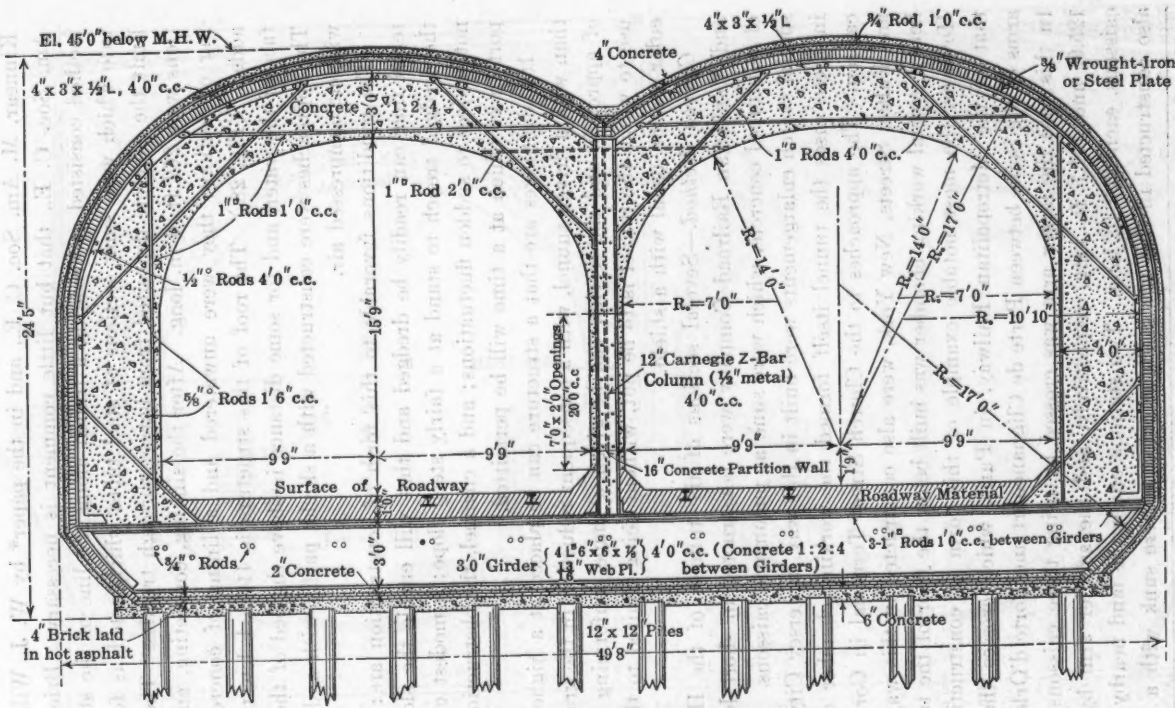
METHOD ADOPTED IN BUILDING THE FIRST OR WESTERN HALF OF THE TUNNEL.

FIG. 25.



PROPOSED HIGHWAY TUNNEL.

FIG. 26.



CROSS-SECTION, PROPOSED HIGHWAY TUNNEL

FIG. 27.

Kinnear, M. Am. Soc. C. E., and in the paper* by W. J. Wilgus, M. Am. Soc. C. E., that but little comment is necessary. Briefly, the method consisted of dredging a trench along the line of the structure into which were sunk sections of steel tubes which acted as forms to limit the concrete deposited from barges through tremies. These sections were 262 ft. 6 in. long. After the sinking, concreting, and joining of sections, they were unwatered and a lining of concrete was added. (Fig. 28.) The roof of this structure is 41 ft. below the surface of the water, and for some distance is above the bed of the river. The approaches were constructed with a shield, partly with and partly without compressed air.

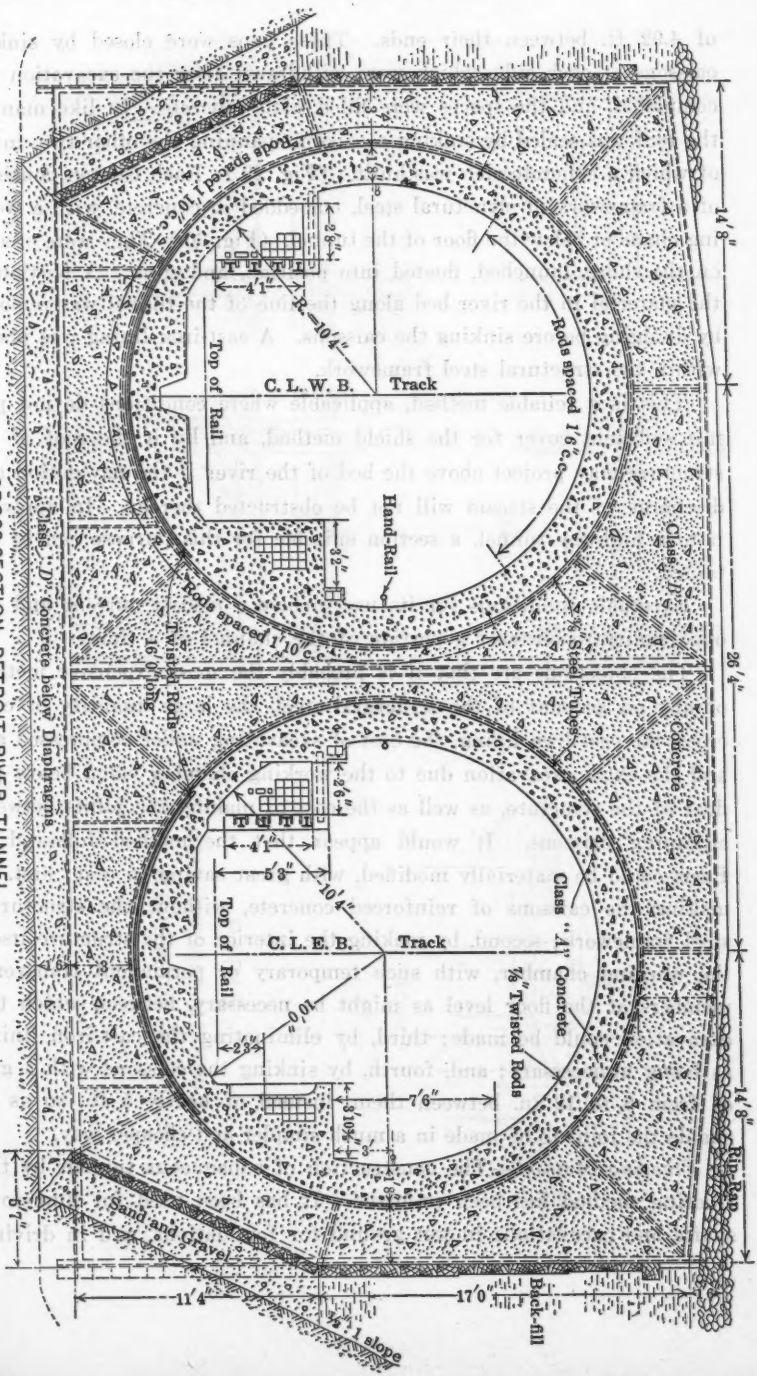
The conditions favorable to this form of construction are: a material that can readily be dredged and that will enable the slopes of the dredged trench to stand at a fairly steep slope; a modest current, not subject to sudden fluctuations; and a channel, the obstruction of a portion of which at a time will be permitted.

Its advantages are that a structure can be placed at a higher level than with a driven tunnel, with a consequent reduction in the gradients of approach, and the section can be given a shape conforming to the use to which the tunnel is to be put, without being limited to the circular section usual with a shield.

Caisson Method.—Several sections of the tunnels of the Hudson and Manhattan Railroad Company were constructed in short lengths of reinforced concrete, which were sunk as pneumatic caissons. Three special switch enlargements were built in this way in Jersey City, and in these cases the tunnel itself formed the working chamber of the caisson. The approaches to the Church Street Terminal in Cortlandt and Fulton Streets, New York, were also constructed in this way, but here a special working chamber was built below the floor of the tunnel. However, the most notable example of this form of construction is that of the Metropolitan Railway, in Paris, which crosses the two arms of the Seine between Porte de Clignancourt and Porte d'Orleans.† In this case the larger arm was crossed by using three caissons, 118, 125.9, and 141.7 ft. long, respectively, and the smaller arm by two caissons, each 65.9 ft. long. Two stations under the land nearby were also constructed in this way. These caissons were sunk with a space

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXV, p. 2.

† "Les Travaux Chemin de Fer Métropolitain Municipal de Paris à la Traversée de la Seine," by L. Biette.



CROSS-SECTION, DETROIT RIVER TUNNEL

FIG. 28.

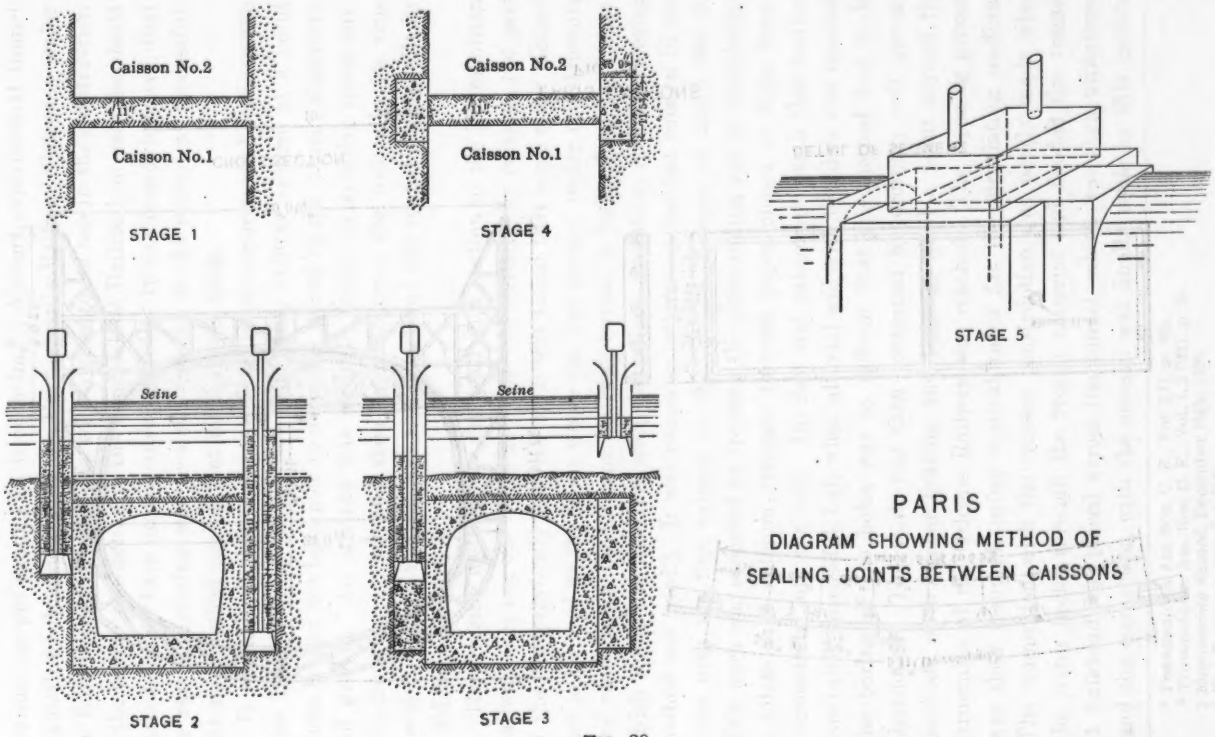
of 4.92 ft. between their ends. These gaps were closed by sinking caissons at each side, which were withdrawn after the excavation was completed, and the spaces were filled with concrete. In like manner, the roof was sealed by sinking a caisson spanning the other two, inside of which a concrete slab was built. (Fig. 29.) Each caisson consisted of a framework of structural steel, embedded in concrete, with a working chamber below the floor of the tunnel. (Fig. 30.) They were erected on the shore, launched, floated into position, and sunk. A portion of the material in the river bed along the line of the tunnel was removed by dredging before sinking the caissons. A cast-iron lining was placed within the structural steel framework.

This is a reliable method, applicable where conditions do not permit sufficient cover for the shield method, and by it the roof of the structure may project above the bed of the river if the navigation and discharge of the stream will not be obstructed thereby. In applying it to a highway tunnel, a section suitable for this purpose can be obtained.

Favorable conditions for its use involve a modest current and the opportunity to obstruct a portion of the channel at a time.

The objections are that it is probably the most expensive method of any on account of the structural steel, the large quantity of concrete, the cast-iron lining, the cost of excavating under compressed air, and the extra excavation due to the working chamber being below the floor of the structure, as well as the cost of making the joints between adjoining caissons. It would appear that the method followed at Paris could be materially modified, with great saving in cost: first, by making the caissons of reinforced concrete, without the structural-steel framework; second, by making the interior of the structure itself the working chamber, with such temporary or permanent transverse members at the floor level as might be necessary, between which the excavation could be made; third, by eliminating the cast-iron lining as being unnecessary; and, fourth, by sinking the caissons with a gap of from 6 to 12 in. between them, instead of nearly 5 ft., so as to enable the joint to be made in a much simpler and cheaper way.

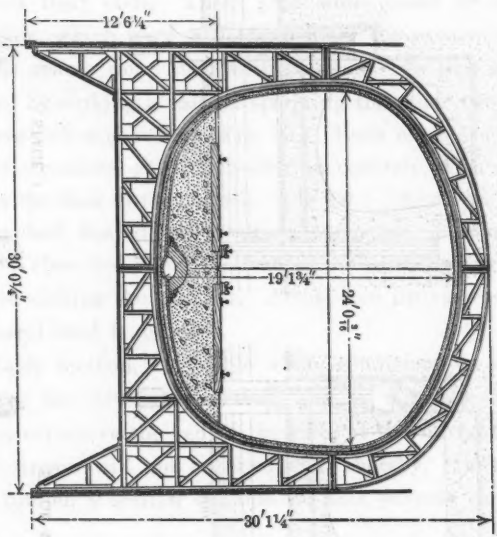
Freezing Method.—The construction of subaqueous tunnels by the freezing method has been proposed. It has been used for numerous shafts and excavations in wet ground for foundations, and in driving



PARIS

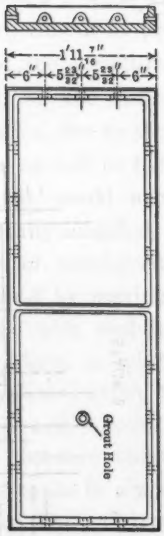
DIAGRAM SHOWING METHOD OF SEALING JOINTS BETWEEN CAISSONS

FIG. 29.

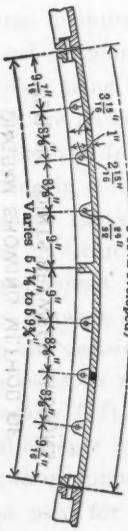


CROSS-SECTION

0 5 10 15 Feet



DETAIL OF SEGMENT



PARIS CAISSONS

Fig. 30.

a tunnel in wet ground in Stockholm.* A short experimental tunnel was built by this method by the Pennsylvania Railroad Company under the East River at New York, with a view to its use in the construction of the tunnel connection to the Long Island Railroad, in case the shield method should have proved unsuccessful.† It was found, however, that the rate of freezing was so slow, and the shield proved so successful, that no practical use of that method was made.

It was also used by the contractor, in an experimental way, on the East River Tunnel of the New York City subway system, at a point where he had not been able to drive the tunnel to the correct alignment and grade.‡ An attempt was made to freeze an annular space surrounding the tunnel and then to reconstruct the lining in its true position, but the method was abandoned and the tunnel was rebuilt by a different method.

The freezing method was also used on a portion of the Metropolitan Railway crossing the Seine at Paris. In building by caissons the part of the line previously referred to, it was found that some other method had to be used at the point where the line crossed under the structure and roadway of the Orleans Railway (which is built along the quay), which would not interfere with the railway operation, and the freezing method was tried.§ It was necessary to connect the last caisson in the river with the first caisson on the land, a distance of about 200 ft. This work was executed by freezing the surrounding soil by circulating a solution of calcium chlorate through pipes placed in holes bored horizontally into the soil. In clay and calcareous earth this method was fairly successful, but, when alluvial soil with boulders was reached, the boring of the holes was so difficult that the method had to be abandoned. The work was then continued by ordinary soft ground methods, with timber bracing and lagging, until a point beyond the structures of the Orleans Railway was reached. The freezing process was then resumed, using vertical tubes for the circulating medium. The connection with the caisson construction was nearly made when the water broke through the frozen material and flooded the tunnel. A bulkhead was placed across the tunnel, which was then unwatered, and the connection with the caisson was finally made by this method.

* *Transactions*, Am. Soc. C. E., Vol. LII, p. 365.
† *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 24.
‡ *Engineering Record*, December 15th, 1906.
§ *The Engineer*, July 1st, 1910.

Notwithstanding the general lack of success with this method, conditions may arise where it would be the most suitable, and it should be considered as one of the resources of the engineer.

Submerged Bridges.—Various projects for submerged tubes, or bridges, have been proposed for crossing bodies of water, the object being to avoid excessive depth and the cost of excavation. Some of these plans are very ingenious, but none has ever been carried out, although many patents on these schemes have been issued.

LINING.

The type of lining is largely dependent on the method of construction. With a circular shield, cast-iron rings with internal flanges are generally used. Cast-steel rings have been used at points where special strength was desired. Considerable thought has been devoted to the substitution of rolled, or pressed, steel segments for the cast iron, but they have been tried only in an experimental way in America. The idea has been that, for a tunnel which must have a lining of masonry, the metal lining should only be considered as a temporary sheeting to enable the permanent lining to be constructed inside thereof and as a water-proofing envelope, and that it, therefore, should be as cheap and light as possible. The tunnel under the Elbe at Hamburg was lined with structural steel with a concrete lining.

With the same object in view, several water and sewerage tunnels have been constructed with a lining of wooden segments, inside of which the concrete lining was built. The approaches to the Detroit River Tunnel were lined with wood in this way.

The Great Northern and City Railway Tunnel, London, was lined with cast iron as the shield was driven, after which the segments in the lower half of the tunnel were removed and replaced with brick masonry; the iron thus saved being used over again.

Brunel's Thames Tunnel was lined with brick masonry, as was a considerable portion of the original Hudson Tunnel, but the latter was not built with a shield.

The original Chicago River Tunnels were also of brick laid in cement, with the outer courses laid in asphalt.

The East Boston Tunnel was lined with concrete and built with a roof shield.

Concrete lining is also essential to the methods used at Detroit and in the subway tunnel under the Harlem River, New York.

Concrete blocks, in lieu of cast-iron lining, for shield-driven tunnels have been proposed and patented by John F. O'Rourke, M. Am. Soc. C. E. (Fig. 31.)*

In a favorable stratum, a subaqueous highway tunnel might be built without any artificial lining at all, but, for sanitary reasons and for ease in illumination, it is doubtful if this would be advisable. The East River Gas Tunnel, at Seventy-first Street, New York City, is about 100 ft. below the surface of the water and is only lined for a short distance in the vicinity of fissures in the rock.

As to the relative advantages of the different linings, cast iron is especially suited to shield construction, and has the advantage of being readily made water-tight, together with its ability to resist the effects of corrosion. For a highway tunnel, it has the disadvantage of being unsuited to any but a circular section. Concrete is readily adapted to a section of any shape. When used with a shield, it must have metal members embedded in it to take the thrust of the shield jacks. When in water-bearing strata under great head, it is difficult to make it entirely water-proof in itself, and it is also difficult to apply an extraneous envelope of water-proofing. A warning should also be sounded in regard to its use in strata containing acid or alkali waters, which rapidly disintegrate concrete. Such waters, though not usual, are not rare. In the first attempt to build the Detroit River Tunnel, sulphur water and sulphur gas were found in large quantities, and sulphuric acid has been encountered on other underground works, so that analyses and tests of underground waters should be made on all tunnel work.

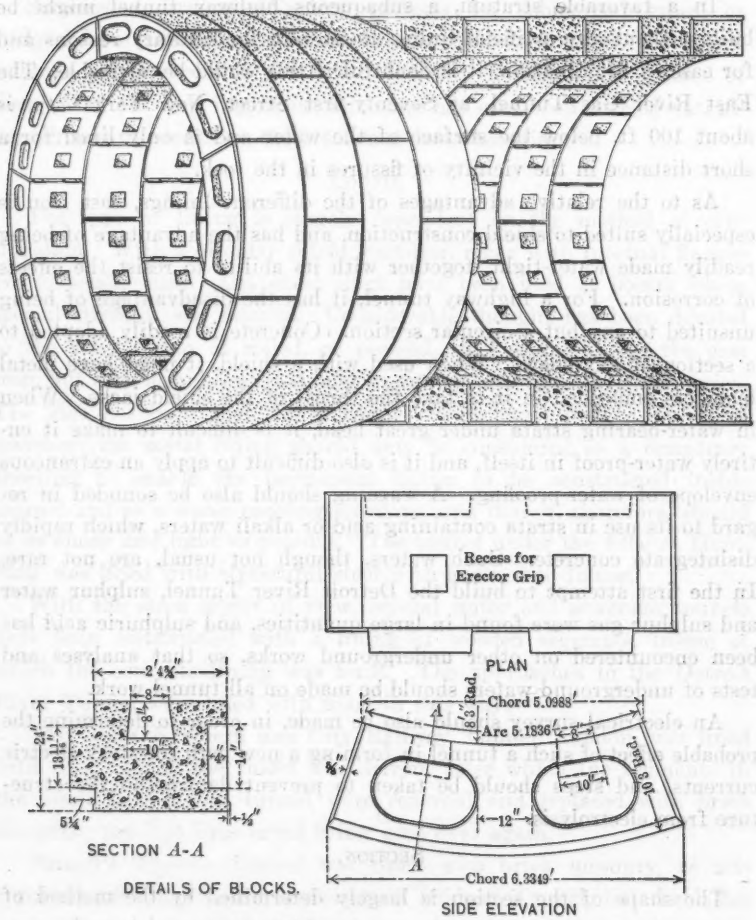
An electrical survey should also be made, in order to determine the probable effect of such a tunnel in forming a new path for stray electric currents, and steps should be taken to prevent damage to the structure from electrolysis.

Section.

The shape of the section is largely determined by the method of construction, rather than by the use to which the tunnel is to be put. Shield tunnels have generally been circular, and this section is not generally well adapted to highway purposes, a semicircular, or horseshoe section being more suitable. The Blackwall Tunnel has a 16-ft. roadway and two 3 ft. 1½-in. footwalks, and the Rotherhithe Tunnel

* *Engineering News*, November 28th, 1912.

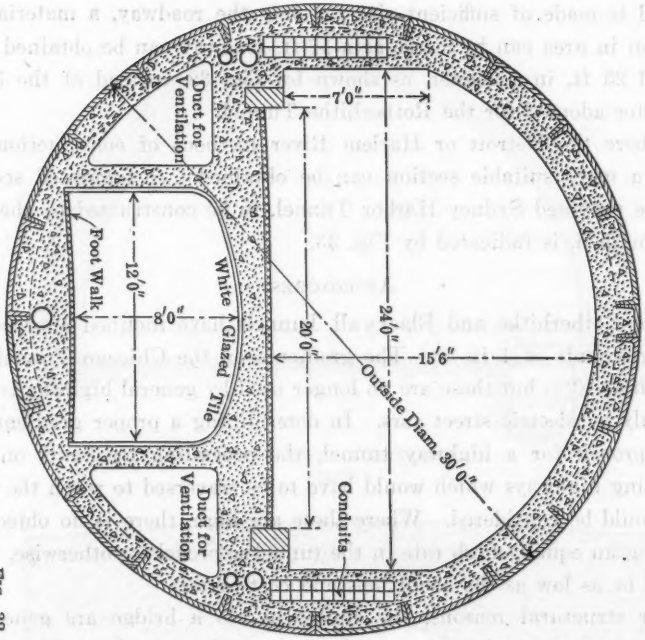
has a 16-ft. roadway and two 4 ft. 8½-in. footwalks. The section indicated in Fig. 32 is suggested as offering a more economical use of the available space, if a footwalk is essential, as it provides a 12-ft. foot-



CONCRETE TUNNEL LINING

FIG. 31.

walk and 20-ft. roadway. In tunnels more than 1 mile in length, it is doubtful if a sufficient number of persons would use the footwalk to warrant its construction, particularly where facilities are provided for crossing in electric cars in the same, or another, tunnel. If the



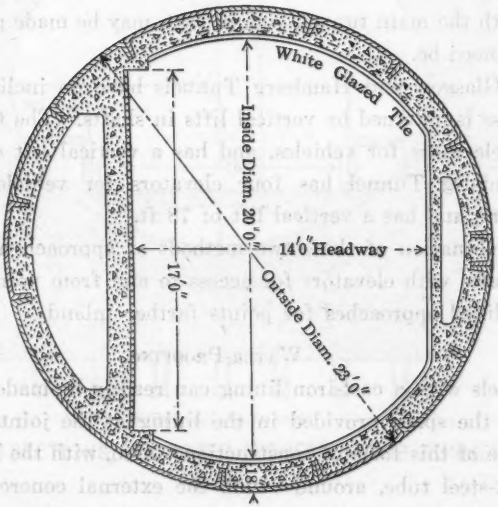


FIG. 32.—CROSS-SECTIONS, PROPOSED NEW YORK HIGHWAY TUNNEL.

tunnel is made of sufficient size for only the roadway, a material reduction in area can be made, as a 17-ft. roadway can be obtained in a tunnel 23 ft. in diameter, as shown by Fig. 32, instead of the 30-ft. diameter adopted for the Rotherhithe Tunnel.

Where the Detroit or Harlem River methods of construction are used, a more suitable section can be obtained. A suggested section for the proposed Sydney Harbor Tunnel, to be constructed by the Detroit method, is indicated by Fig. 33.

Approaches.

The Rotherhithe and Blackwall Tunnels have inclined approaches, with gradients of 1 in 36. The gradients on the Chicago Tunnel are as high as 10%, but these are no longer used by general highway traffic, but only by electric street cars. In determining a proper gradient for an approach for a highway tunnel, the prevailing gradients on the adjoining highways which would have to be traversed to reach the tunnel should be considered. Where these are high, there is no objection to using an equally high rate in the tunnel approaches; otherwise, they should be as low as the conditions and cost permit.

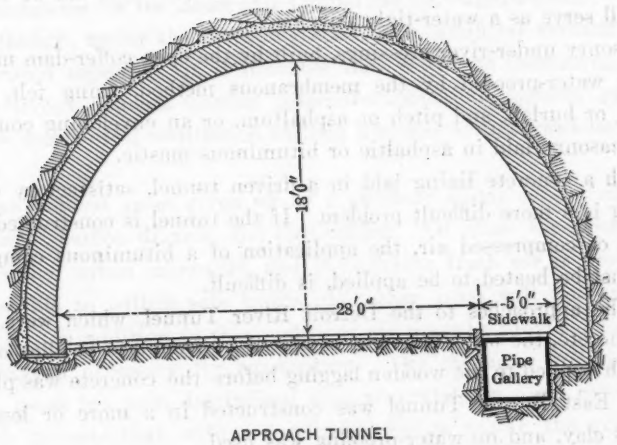
For structural reasons, the approaches to a bridge are generally made on a prolongation of the axis of the main span, but with a tunnel this is not necessary, and they may be curved or at any convenient angle with the main tunnel, and, in fact, may be made parallel with the river, if need be.

The Glasgow and Hamburg Tunnels have no inclined approaches, but access is obtained by vertical lifts in shafts. The Glasgow Tunnel has six elevators for vehicles, and has a vertical lift of about 65 ft.; the Hamburg Tunnel has four elevators for vehicles and two for passengers, and has a vertical lift of 78 ft.

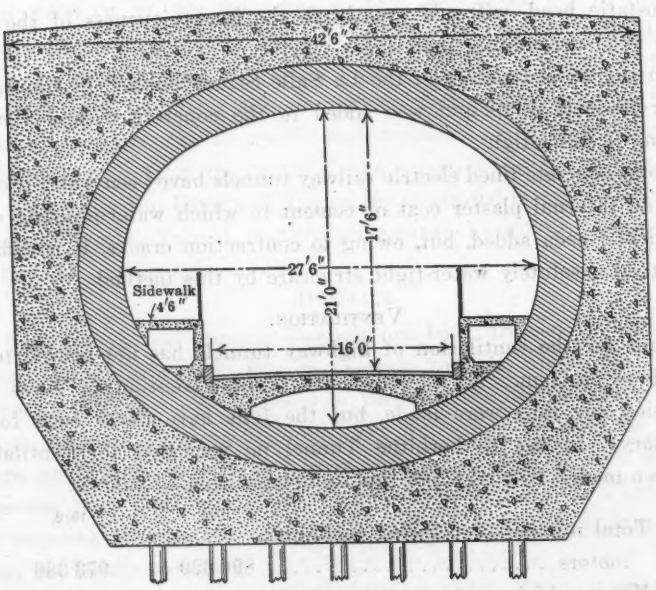
A combination of these two methods of approach may be made by using shafts with elevators for access to and from points at the shore, with inclined approaches for points farther inland.

Water-Proofing.

Tunnels with a cast-iron lining can readily be made water-tight by caulking the space provided in the lining at the joints, which is one advantage of this form of construction. Also, with the Detroit method, the sheet-steel tube, around which the external concrete is deposited, can be caulked and will keep the tunnels tight. Tunnels sunk by the



APPROACH TUNNEL
(ROCK)



SUBAQUEOUS TUNNEL

CROSS-SECTIONS, PROPOSED SYDNEY HARBOR TUNNEL

FIG. 33.

caisson process can have an inner or outer shell which can be caulked and will serve as a water-tight envelope.

Masonry under-river crossings, built by the open coffer-dam method, can be water-proofed by the membranous method, using felt, duck, canvas, or burlap, and pitch or asphaltum, or an enveloping course of brick masonry laid in asphaltic or bituminous mastic.

With a concrete lining laid in a driven tunnel, satisfactory water-proofing is a more difficult problem. If the tunnel is constructed with the aid of compressed air, the application of a bituminous compound that must be heated to be applied, is difficult.

In the approaches to the Detroit River Tunnel, which were lined with concrete, the water-proofing consisted of from 3 to 11 plies of felt and pitch applied to the wooden lagging before the concrete was placed.

The East Boston Tunnel was constructed in a more or less impervious clay, and no water-proofing was used.

The liberal use of grout at a greater pressure than that due to the hydrostatic head, will add greatly to the imperviousness of the concrete and surrounding material.

In the reconstruction of the La Salle Street Tunnel, in Chicago, a water-proofing compound was added to the concrete with a view to making it water-tight.

Some concrete-lined electric railway tunnels have been water-proofed with an internal plaster coat of cement to which water-proofing compound had been added, but, owing to contraction cracks, it is difficult to get an absolutely water-tight structure by this method.

Ventilation.

Thus far the ventilation of highway tunnels has not proved to be a serious problem. In the Blackwall Tunnel ventilation ducts and provision for fans were made, but the fans have never been found necessary. In the Rotherhithe Tunnel no provision for ventilation has been made. The traffic in this tunnel has been as follows:

	1911.	1912.
Total number of vehicles, including motors	896 629	973 336
Motor vehicles only	21 008	26 998
Maximum number of vehicles passing through the tunnel in one hour		325

Of the latter, 10 were motor-driven.

The figures for the Blackwall Tunnel are somewhat similar.

Although, under the foregoing conditions, artificial ventilation has not proved necessary, it is probable that, with the increasing use of motor-driven vehicles having internal-combustion engines, provision for the mechanical ventilation of long highway tunnels will have to be made.

The exhaust gases given off by a gasolene motor consist of carbon monoxide, carbon dioxide, and free oxygen. Of these, the most poisonous is the carbon monoxide, and if sufficient fresh air is provided to dilute this to within safe limits, the other gases will not cause any trouble. In addition to this, sufficient air must be provided to furnish the oxygen necessary for combustion. The character and quality of these gases has been determined very accurately in some recent tests* which indicate that, with a 6-cylinder, 48-h.p. motor, the quantity of air required to dilute the gases given off, within safe limits, would be about 8 000 cu. ft. per sec. when in first gear, 4 000 cu. ft. per sec., when on second gear, and 2 000 cu. ft. per sec., when on third gear. Based on these figures, it will be found that the ventilation of such a tunnel, even assuming that motor cars follow each other as closely as safety permits, can be readily accomplished. It can be effected by withdrawing air from the center of the tunnel through conduits under the floor, as in the Blackwall Tunnel, or above the clearance line, as in the East Boston Tunnel, or through separate ventilating passages, as in the Mersey Tunnel.

Drainage.

The Severn and Mersey Tunnels are both drained by letting the water leaking into them flow in separate tunnels, constructed for the purpose, from the lowest points in the main tunnels to shafts on the shore, where it is removed by pumping. In both these cases the tunnels were constructed in rock and lined with brick masonry, and the leakage was very large, but in more recent tunnels, lined with iron, steel, or concrete, the leakage is so small† that such special drainage headings are hardly necessary. In these latter tunnels the water is removed by pumping from the sumps through pipes laid along the sides of the tunnel to the shore.

*"A Comprehensive Motor Test," by Herbert Chase, *Transactions*, Soc. of Automobile Engineers, 1912, p. 40.

† Detroit Tunnel, *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 349; Hudson Tunnel, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI, p. 169.

The art of boring large-sized holes in any direction has developed to such an extent that there is no reason why a hole cannot be bored from the sump upward through the overlying material to the bed of the river through which to discharge the drainage and thus save the long length of pipe to shore and the increased friction head.

Conclusions.

In this paper the writer has endeavored to assemble the published facts relative to all the subaqueous highway tunnels in the world, as well as of subaqueous tunnels built for other purposes, where the methods of design or construction would have an application for highway purposes. The paper, therefore, is largely a compilation, the information having been obtained from the *Transactions* of this and other engineering societies, and from the technical press. Credit for this information is generally given, but in a number of cases where the information came from various sources, the authority is not stated, and the writer wishes to take this opportunity to express his indebtedness to those who have written on this subject.

The need for improved methods of vehicular communication around harbors is manifest, and it would seem that the furnishing of this by highway tunnels has not received the attention it should. This paper, therefore, has been prepared in the hope that it will develop a discussion which will stimulate attention to tunneling for this purpose.

Commercial transportation was formerly largely conducted on the highways, and the world will shortly complete a century of railway transportation, one of the consequences of which was neglect of the highways, but the present tendency is to return to the highway for short-haul transportation, largely owing to the development of the motor vehicle for pleasure and business, which is leading to a revival in the study of road construction to meet new conditions, and highway tunnels, in many localities, are a necessary link to make this system of communication continuous.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

BY MESSRS. J. N. DODD, MORRIS KNOWLES, M. O. LEIGHTON, J. P. SNOW, ARTHUR L. MILLS, J. M. SCHREIBER, J. P. NEWELL, GARDNER S. WILLIAMS, GEORGE F. SWAIN, J. W. LEDOUX, ALBIN G. NICOLAYSEN, J. B. LIPPINCOTT, ALFRED NOBLE, M. L. BYERS, AND FREDERIC P. STEARNS.

J. N. DODD, ESQ.—The general principle involved in valuation for rate-making has been stated very clearly in the report, namely, that the primary requirement is that justice be done both to the owner of the property and to the public, and that the owner is "entitled to earn a fair return upon a fair value of the property utilized in or reasonably necessary to the service". This has been settled by the Courts, and no dissent can be taken from such ruling. On the other hand, as pointed out by the Committee, no complete or definite rule has been laid down for the determination of the fair value. Mr. Dodd.

It is becoming generally agreed that, as regards what are called public utilities, the public, as a rule, is better served by a monopoly than by competition. One gas company can manufacture and distribute gas more cheaply than six companies. The distribution by one company can be made more satisfactory, both to the consumer and to the public at large, than by six. For example, the disturbance due to tearing up the streets is far less when one company produces and distributes than when there are six.

On the other hand, the dangers of a monopoly are well realized. One of the functions of the Legislature, now usually delegated to the public service commission, is to guard against such dangers and to protect the consumer against unreasonable demands on the part of public utilities.

* Continued from April, 1914, *Proceedings*.

Mr. Dodd.

For future and for recent public service properties, the Committee favors the actual cost as the fair value. This is apparently the value to be given to the property for all time to come. For old public service properties, the Committee favors the cost of reproduction less appreciation of the identical plant at present-day prices of labor and material, mainly because it is claimed that it is impossible to determine the actual original cost of such properties.

Underlying the principle that the fair value for rate-making purposes is the "actual cost of the identical plant", is the theory that the owner of the property has a right to a fair return on the money actually invested. Such a theory has much to commend it and is widely held. Among the theories of valuation for rate-making, there is at least one other for which strong arguments can be made.

The advantages of competition are real. In ordinary business it is only when there are no restrictions, when trade is free, and when competition is unrestricted that the community is best served, that prices are lowest, that the ingenuity of manufacturers is stimulated to design the best machines, and that the capitalization of the competing concerns is kept down to a healthy figure.

A method of valuation for rate-making, which would retain for the public the advantages of monopoly and, at the same time, obtain for them the advantages of competition, would be ideal from many standpoints. It would insure to the public the advantage of the most modern machinery and the most approved methods. It would also insure to the company the full increase in the value of its properties and plant caused by the increase in the size and prosperity of the community. This latter increase is in part due to the service of the public utility corporation and, therefore, the corporation is fully as much entitled to it as the householder to the increased value of his house or lot, or the storekeeper to the increased value of location due to the improved character of the neighborhood developed around his store.

In a word, the purpose of valuation should be to restore values to a competitive basis. The rules governing the management and operation of a public utility company should differ very little from those of any other well-managed business. The community gives the company a certain monopoly. In return it has a right to all the improvements in the art and to sound business management. The city should pay prices which permit of ample allowance for depreciation and development and are commensurate with those in other lines of business.

If in ordinary business a man makes a mistake and invests in a machine which is useless or which shortly becomes obsolete, the community suffers very little. It is true that a certain amount of capital has been lost, for which, eventually, the entire community must pay,

Mr. Dodd.

but, on the other hand, the community does not continue to pay interest on the investment, nor does it continue to pay high prices for goods produced by an obsolete process.

Competitors are always ready to take advantage of all improvements and to undersell those who have made unlucky investments. Improvements are eagerly hailed, because they actually represent a gain in the wealth of the entire community, in spite of the fact that such improvements mean a serious loss to some person or group of persons.

Speaking generally, therefore, the valuation of a public utility company should be the cost to reproduce a substitute plant, such a plant as a competitor might build, not a valuation of the identical plant.

With this general plan to guide, namely, that the valuation is desired of the plant which a competitor would build to give the same service, it becomes possible to determine not necessarily all the steps of the valuation, but, in general, what is desired.

The value of the machinery should be that of the latest improved kinds to give the same service. The cost of the real estate should be its present-day cost. The value of the buildings should be their value as if built according to present-day prices of labor and material. The value of underground pipes and conduits should be the value according to present-day prices of labor and material and present types of pavements.

On the other hand, the unit prices should not be those of to-day nor of yesterday, but the average prices for a number of years past.

As regards the question of when it is satisfactory to value the identical plant and when the valuation should be of the substitute plant, the same rule of liberality should be adopted as that recommended on page 14 of the report, in regard to unused property. There is no question that a modern steam turbine is far cheaper and more efficient than a slow-speed, reciprocating engine of the type in favor, say, 10 or 15 years ago. There is no question as to the comparative advantage of a modern water-gas producer over the old type of gas retort. There is usually no question regarding the comparative suitability of three 8-in. gas mains or a single 14-in. main. When there is no question that the machines actually installed are obsolete, inefficient, and expensive, the valuation should be of a substitute plant; however, when there is room for question, it should be that of the "identical plant."

With respect to the value of real estate and of buildings, there seems to be little difference between what is recommended in the report as regards the valuation of old properties and the method advocated by the speaker for the valuation of all properties.

As to paving over conduits and pipes, it is as much an increment to the value as the development of real estate near a railroad track, an addition to the cost to reproduce the road, and a just reason for in-

Mr. Dodd.

creasing the taxes on the road. The paving adds to the cost if the competitor wishes to build; so also do the buildings restrain competitive building, in the case of a railroad. It is true that the company pays nothing for the cost of the paving, but, on the other hand, neither does the railroad pay for the buildings. Each case is a natural result of the growth of the community caused in part—sometimes in large part—by the company.

The full increase in the cost of the mains due to the paving should not be added to such cost, but only a proportion, depending on their age and estimated life. The cost of installing a competitive line is the same, regardless of the time when the paving was laid down, whether a year before or 20 years before. Therefore, if, for example, 80% of the estimated life of the main has expired at the date when the street was paved, only 20% of the added value due to the paving can be added to the capitalization of the company. In the same way, if the estimated life of the mains is 20 years, the annual depreciation which can be charged on the paving is the same percentage as on the mains, namely, 5% of the total additional cost to reproduce due to paving.

Although the report is notable for its clearness and for the many valuable contributions it has made to the science of rate-making, now in its formative stage, it appears to be open to criticism in one respect, namely, that it is difficult to determine the ground principles which underlie the proposed method of determining the cost to reproduce.

In general, it favors the actual cost of the identical plant. In the case of an old property (page 9), a deviation from this is recommended, because it is claimed to be impossible to determine the actual cost. It is difficult to accept the reason. There seems to be no insuperable difficulty in the way of determining the original cost of the labor and material of the identical units now in service.

For unused property, the report shows (page 13) that this rule is generally abandoned by the Courts, and that such property cannot be included, either at the original, or at any other cost.

As regards the present or original condition of the land (page 15), the Committee recommends that valuation should be based on the physical conditions existing at the time the property was built, but at present-day prices. In other words, it abandons wholly the idea of original cost.

When property has been donated (page 18), present-day prices should be used. Here, also, the principle is violated.

With regard to unit prices (page 30), the Committee recommends that present-day prices should be used for labor and material. The reason for this is not because of the difficulties attending the determination of original prices, but because Court decisions are in favor of present-day prices.

Mr. Dodd.

It appears, therefore, that the principle of permitting the company to earn on the actual original cost of the present property is one which the Courts have apparently refused to uphold; but, on the contrary, in all Court rulings to which reference is made in the report the present-day cost is the only one which has received their sanction (page 20).

The Courts also generally refuse to sanction unused or abandoned property as being included in the assets (page 13). Is there good reason to doubt that they will consider obsolete inefficient apparatus in the same manner, and insist on a valuation of such machinery on a basis of the service rendered?

The Courts have granted the companies the right to the full increase in the value of the real estate, caused by the increase in the wealth of the community. They have granted the companies the same right to an increase in the value of buildings. It seems fair to assume that the Supreme Court will fail to grant a similar increase in the value of the mains as affected by the pavement.

It is believed that the method offered herein of valuing a plant on a competitive basis, that is, purely present-day prices of a substitute plant, would offer fewer inconsistences and would be more permanently satisfactory.

Mr. Knowles.

MORRIS KNOWLES, M. AM. SOC. C. E.—It is not to be presumed that the Committee's report, on a question still so much in process of development, can at any time, now or after much discussion, result in the complete agreement of the membership, or in the expression of any conclusive opinion by the Society. Members will gain, however, by the free interchange of opinion, will have their ideas clarified, and may be able to agree on some few fundamental principles. The correct application will continue for a long time to appear elusive. To aid in the discussion and understanding of the report, a tabulated list of definitions, or glossary, of the important terms used would be of great help, and it is to be hoped that the Committee will add such a list before concluding the report.

The speaker's remarks relate largely to public utilities of restricted extent, with which he is more familiar, and not to railroads, because he can readily realize that the principles, at least for the present, that apply to such a utility, reaching to many different places, affecting many different peoples, and so many kinds of service, may well be different from those applying to a regulated monopoly in a narrow field.

Purpose of Valuation.—The Committee limits its report, by title and by explanation in the opening paragraphs, to the subject of "Valuation for the Purpose of Rate-Making", and states (page 5) that valuations for different purposes may not give identical results, "if the laws of the different States and certain decisions of the Courts are complied with".

Mr. Knowles.

In another paragraph (page 6), the Committee states:

"It is recognized, however, that the whole subject is in a developmental stage, and while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

To the speaker there seems to be a certain degree of inconsistency in these statements. It is true that the subject is in a developmental stage and that the laws and decisions are constantly changing. It is common knowledge, also, that many of the recent decisions on these questions have been strongly influenced by consideration of expert testimony by engineers. May we not hope that a still larger influence in this direction will be enjoyed by this Society and its members? And, under the present conditions, would not the Committee accomplish its greatest usefulness by considering this, as it does other topics, from the point of view of sound principle, attempting to formulate true ideals, and by recommending practices which approximate these ideals as closely as existing laws and decisions permit? When authorities differ so widely, this would seem to be much more preferable than merely recognizing existing conditions and practices, and attempting to formulate ideals from them.

President Wilson has aptly stated "the day of accommodation, of concession, and of common understanding is, of necessity, a day of achievement". We are rapidly coming to the common understanding that fair return means equity, that it has all the force of its literal significance, alike to server and to user, *viz.*, an equality in value between the wealth, labor, and effort devoted by the owner to the public use, and the money returned to him by the public in rates. When this has finally been thoroughly established, a consideration of the matter in the light of principle will lead, the speaker believes, to the conclusion that valuations for all purposes, under complete and comprehensive regulation, should be identical.

It is true that no useful purpose would be served now by attempting to insist under the present laws that valuations for taxation be identical with other valuations, but the reason for this is that the problem of valuation for taxation is not a problem in valuation at all, but one in taxation. The method of valuation for taxation is fixed by the tax laws, and not by any consideration of the true meaning of valuation. The speaker believes that when an ideal system of taxation has been worked out, if it is based on property valuation at all, and requires valuation of public utilities for taxation, such valuations will be identical with those for purchase, capitalization, and rate-making. Until that time, the varying requirements of taxation laws cannot be adduced as throwing any light on the subject of valuation.

Mr. Knowles.

The following statement is made, on page 8 of the report:

"As the future earning capacities of such properties under rate regulation by commissions will to a large extent depend upon rates to be established as a result of valuations of the properties, it is clearly illogical to base the valuations on present or past rates. Such valuation would be reasoning in a circle."

If such be the case, and if in the future the purchaser will be held down, and if he cannot earn on a different or greater value than that fixed by public regulation for rate-making, where will be the incentive to pay more for the property; what will keep alive the spirit of gambling in the future?

If it is agreed that a fair return is to be permitted on everything that the owner of a public utility has devoted to the public service, then the value for rate-making purposes will be the total actual reasonable investment, including all preliminary and development expenses and every sacrifice of time and labor, as well as of money. If returns on this value are limited to the market rate of borrowed capital under the given conditions of risk, then the value of the property for purchase would be that return capitalized at the same market rate of interest, or a value identical with that for rate-making purposes; and, if capitalization is to be controlled, so as to represent the actual investment, either of the original owner or of the purchaser, then the valuation for capitalization will be identical with those for rate-making and purchase.

The speaker then, while recognizing the laudable desire of the Committee to limit its report to one subject at a time, suggests that it is unwise to place on record, even by implication, that, either for present legal reasons or otherwise, valuations for different purposes will necessarily give different results. It would seem as if the Committee could be of more service by insisting throughout on the principles involved, instead of compromising, as in the statement first quoted above, with existing laws and decisions of Courts.

Present Conditions to Govern.—The report recommends: "the method of valuing the substantially identical property" according to "original conditions"; but goes on to say "valuations should be based upon the physical conditions at the time the various portions were built, but at the prices prevailing at or near the time of the valuation." This is manifestly an inconsistency, and an unnecessary straddle.

Let us analyze the attempts on the desire to do justice to the owner and rate-payer: It is generally recognized, as so ably stated by the Committee, that original cost, to include all proper items, and elements, and percentages, will give the desired result, if obtainable. It is rare that this can be determined, and this has led to the method—born of necessity—of estimating the cost-of-reproduction-new; determining

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all factors as of the time of valuation, with normal prices for the preceding few years; but this should never be considered an end in itself, but only a means of throwing light on the probable original cost; and, if this is kept in mind, there will be no excuse for confusion arising from doing partly one thing and partly another. Granted that full historical and book cost may be difficult, even impossible of complete determination, and that, better than present-day-cost-of-reproduction is the cost-of-reproduction-with-original-conditions; the speaker submits that if "original conditions" can be determined, or even approximated, it is also true that prices at the time of construction can be even more accurately found, as these are frequently of record in the engineering and business periodicals, and in published reports and contracts of the time of construction. By using these we shall thus have harmony and consistency of thought and purpose.

Depreciation.—The Committee's statement on this subject is perhaps the most novel presentation in the report, and will arouse the greatest discussion. Time has not permitted an adequate analysis of the supplemental statement on this subject, but just a word may not be amiss. We have heard a great deal of the equity of charging rates upon 100% of cost new, on the basis that the service is kept up, that it is of full worth and therefore should be paid for accordingly, no matter what is done with the depreciation fund. This, we are told, is the concern of the stockholders. Now, much of the confusion and disappointment in the past have come from the endeavor to secure from these same stockholders the full repayment to keep up plant and service; and these stockholders are too frequently the "ubiquitous widows and orphans", who have acquired their stock from original owners who "got out from under" after drawing dividends at the expense of the depreciation fund.

Perhaps it would help us understand the position, so ably presented by John W. Alvord, M. Am. Soc. C. E.,* when he says the value for calculating rate of return depends on whether the fund has been kept intact, if we would remember not only that this fund, made up by annual payments by whatever system, is due the owners, that is, shall be paid in each year by the rate-payer, but also that the owner must preserve this fund intact for the purposes for which it was created. That is, it is a trust fund, and the owner has all the solemn duty of a trustee to preserve the fund for use in keeping up the property for the user who has paid into this fund.

It may even be true, in some cases of stagnant property and in order that refunding of bonds at maturity may be possible, that some small payment to a sinking fund as well may be necessary. This has been hinted by the Committee on page 7, when it says:

Transactions, Am. Soc C. E., Vol. LXXVII, p. 788.

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"It is not enough * * * that the borrower should pay interest on the money loaned, but he must also pay the principal to cancel his indebtedness."

TABLE 19.—STATEMENT OF ENGINEERING COST ON CONSTRUCTION, BETWEEN APRIL 1ST, 1904, AND JANUARY 31ST, 1910, ON PITTSBURGH FILTRATION WORKS.

Contracts.	Final estimates.	Engineering Cost.	
		Total.	Percentage of final estimates.
1. Slow sand filters, basins and appurtenances	$3 359 161.49	$332 854.65	9.91
2. Filtered water reservoir and appurtenances	415 245.28	46 398.07	11.18
3. River crossing and connections at Brilliant	266 474.36	27 944.26	10.47
4. Centrifugal pumps, engines, generators and appurtenances, at Ross pumping station	92 956.30	4 385.76	4.72
5. Boilers, economizers, piping and appurtenances at Ross pumping station	68 403.53	2 676.59	3.94
6. Sand washers, electric machinery and auxiliary equipment for Ross pumping station	122 568.21	11 804.29	9.60
7. Ross pumping station	356 580.69	41 324.24	12.58
8. River wall, intakes and connections and grading, pumping station and machinery foundations	208 318.92	26 933.76	12.94
9. Pumping engines, boilers and appurtenances at Brilliant pump station	218 028.37	9 767.27	4.48
10. Pipe line and appurtenances, Highland Reservoir No. 2 to South Side	555 250.76	34 877.71	6.28
	$5 662 987.91	$538 966.60	9.52

TABLE 20.—FINAL DISTRIBUTION OF CONTRACTOR'S COSTS BY BUREAU OF FILTRATION, ON PITTSBURGH FILTRATION WORKS.

	Contract No. 1. Slow Sand Filters and Appurtenances.		Contract No. 2. Filtered Water Reservoir and Appurtenances.		Contract No. 3. River Crossing and Connections at Brilliant.	
	Amount.	Percentage of total.	Amount.	Percentage of total.	Amount.	Percentage of total.
Materials and sub-contract items	$1 733 378	47.2	$165 758	35.3	$105 567	35.0
General supplies	127 402	3.4	16 087	3.4	10 327	3.4
Plant	275 249	7.5	34 766	7.4	22 318	7.4
Pay roll	1 164 613	31.8	206 113	43.9	133 572	44.3
General expense	254 672	6.9	32 156	6.8	20 644	6.8
Interest and general overhead charge	115 250	3.2	14 570	3.2	9 342	3.1
	$3 670 564	100.0	$469 450	100.0	$301 770	100.0

Engineering and General Expenses.—To those who have viewed with wonder the generally accepted basis for costs of engineering services of 5% and 6%, it is gratifying to note the results of the Commit-

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tee's inquiries into such costs on great and important water-works and transit systems, revealing amounts of 8 and 10 per cent. The speaker is glad to add one more set from the records of the Pittsburgh Filtration Works, from 1904 to 1910, inclusive, in Tables 19 and 20.

Extra Capacity.—The position of the Committee in regard to valuing all the property, including reasonable allowances for increased size, capacity, and availability for the future, cannot be too earnestly commended. No one with judicious foresight should be held down to designing works for the present only. Reservoirs, pipe sizes, filters, buildings, pumps, all need to be planned in varying degrees for the probable needs of a number of years. This is but ordinary good business judgment, and the sooner we can get the Courts to recognize this principle the better for all.

Mr. Leighton.

M. O. LEIGHTON, M. AM. SOC. C. E. (by letter).—The writer will participate in this interesting discussion only to the extent of calling attention to an interesting Court decision with reference to a much disputed point in the determination of "replacement value".

Mr. Allen Hazen, in his discussion of this report, makes the following statement:

"* * * in estimating the cost of reproduction of a water-works plant, the cost of cutting through the paving of the street over it, and replacing such pavement after laying the pipe, is unquestionably a part of the cost of reproduction, and must be included in any true estimate of it. The Committee, however, would exclude all that part of the cost which relates to paving laid after the pipe was placed. The writer is unable to see that the question of whether the pipe or the paving was laid first has anything to do with the cost of the reproduction of the property as it now stands, or with its value. It would cost just as much to replace the pipe in one case as in the other. The value of the property, as measured by its capacity for useful service, is identical in both cases."

On March 24th, 1914, the Court of Appeals of the State of New York, handed down an opinion in the case of the Kings County Lighting Company *v.* the Members constituting the Public Service Commission of the State of New York for the First District. Among the questions certified for decision were the following:

"Was the relator entitled, upon the facts shown in the record, to have the cost of reproduction of paving now in the streets but not in place at the time the mains were laid, allowed for in ascertaining the value of its property used in the public service?"

"Was the relator entitled, upon the facts shown in the record, to have the cost of reproduction of paving now in the streets but not in place at the time the mains were laid, allowed for in ascertaining the capital actually expended?"

Mr. Leighton.

With reference to the two foregoing questions, the Court said:

"In determining the cost of reproduction, the Commission allowed $12 717 as the cost of restoring the pavement as it existed when the mains and service pipes were laid in the streets. The relator claimed an allowance of at least $200 000 for the cost of restoring pavements subsequently laid, on the theory that that cost would have to be incurred if the mains were to be laid to-day. But the new pavement in fact added nothing to the property of the relator. Its mains were as serviceable and intrinsically as valuable before as after the new pavements were laid. The controlling considerations under the preceding point also determine this. The rights of the public are not to be ignored. The question has a double aspect. What will be fair to the public as well as to the relator? (Smyth *v.* Ames, *supra*). Should the public pay more for gas simply because improved pavements have been laid at public expense? It is no answer to say that the new expensive pavements suggest improved conditions which, though adding to the value of the plant, will not, by reason of the greater consumption, add to the expense per thousand feet of the gas consumed. The public are entitled to the benefit of the improved conditions, if thereby the relator is enabled to supply gas at a less rate. The relator is entitled to a fair return on its investment, not on improvements made at public expense. It is said that the mains will have to be relaid. So will the new pavements, and much oftener. Both might possibly be relaid at the same time. The case is not at all parallel to the so-called unearned increment of land. That the company owns. It does not own the pavements, and the laying of them does not add to its investment or increase the cost to it of producing gas. The cost of reproduction less accrued depreciation rule seems to be the one generally employed in rate cases. But it is merely a rule of convenience, and must be applied with reason. On the one hand, it should not be so applied as to deprive the corporation of a fair return at all times on the reasonable, proper, and necessary investment made by it to serve the public, and, on the other hand, it should not be so applied as to give the corporation a return on improvements made at public expense which in no way increase the cost to it of performing that service."

Mr. Snow.

J. P. Snow, M. Am. Soc. C. E.—The following has been suggested by the printed discussions of the report which have so far been issued.

In formulating its report, the Committee assumed certain fundamental principles to be axiomatic which several critics of the report have considered open to discussion. An explanation of a few of these will render the discussion clearer.

First.—It is claimed by some that valuations cannot be used in fixing rates for public utilities, especially for steam railroads. It seems idle to make this claim, in view of the facts that an Act of Congress provides for the valuation of railroads, and the Interstate Commerce Commission and its engineers state that valuations are an important factor in fixing just rates, and must be ascertained at whatever cost for their information and use. The railroads of the country are preparing

Mr. Snow.

to make these valuations, and the Government engineers are organizing their forces to check the work. Where, then, is the force in the argument that these valuations cannot be used, and that the Committee should not have considered valuation for rate-making purposes in its report.

Second.—Several of those who have discussed the report claim that, because an item of property is rendering efficient service, although it is confessedly near the end of its life, it should be scheduled as having value new. This idea rests on the theory that rates should be based on the value of the service to the consumer, instead of on the cost of the service to the producer. Although aware that in the last analysis the law of supply and demand rests on a balance of these two theories, the Committee is firmly of the opinion that the value of service to the consumer cannot be used as the basis for computing rates in terms of money. Property must be kept in a state of full efficiency; else rates cannot be collected at all. When an item reaches the stage where its efficiency is practically impaired, it must be replaced and either scrapped or relegated to other parts of the service where less efficiency suffices. It follows that continued efficiency cannot be considered as a reason for holding an item in the schedule at value new throughout its term of life.

Third.—Protection of the investment seems to be another stumbling block with many who have discussed this question. The aim of the Committee has been to protect the owner in the legitimate, proper value of his property, based on present prices and original conditions so far as they have largely controlled the cost and are definitely known. If investment *per se* is to be considered, shall it be the original cost of an old property, or the cost to the present owners? Is the cost entirely legitimate? Has there been appreciation of value in some part of the holding? Has some part of the property been acquired by gift? These and many other questions render a full dependence on investment cost impossible and unjust—unjust sometimes to the consumer and sometimes to the owner. Justice seems to require that original conditions be considered as above, especially as regards the improvements on real estate, paving over pipes already laid, etc., because the legitimate, proper cost, so far as ascertainable with certainty, plus appreciation and minus depreciation, is the value. Higher present unit prices are an item of appreciation; and lower, of depreciation. Paving over pipes is a factor of added cost when replacement occurs, plainly chargeable at that time to capital as betterment, under the Interstate rules for railroad accounting.

Although the consideration of original conditions to any extent seems to hark back to that of investment, it does so only in so far as strict justice, to the owner on the one hand and to the rate-payer on the other, demands.

Mr. Snow.

It must not be supposed that hard and fast rules can be adopted for solving the problem of valuation for rate-making purposes, and can be extended to cover abnormal cases. Tolls on the Panama or New York Canals cannot be fixed by the Committee rules any more than can the price of diamonds from the Kimberly Mines. The application of any rules on the subject must be tempered by common sense, and this necessity has been kept in mind at all times by the Committee.

Referring to the discussion by Messrs. George F. Swain, D. W. Lum, and others: The word "rate", as used by the Committee, should be assumed to mean the gross income from the physical property which is valued; and does not go into the intricacies of a schedule of individual rates like a railroad tariff sheet.

The difference between using interest return on value new plus a sinking-fund allowance, as advocated by George F. Swain, Past-President, Am. Soc. C. E., and the Committee's recommendation of interest on depreciated value plus a depreciation allowance, depending on the life of an item and its age, lies more in the bookkeeping than in the amount of money involved.

Mr. Swain's is the easier method for the engineer, in that only the value new of the item and its term of life are needed, while the other method requires in addition the age of the item. The objections to his method are serious, however, and apparently very practical. The speaker begs leave to differ with him by thinking that the sinking-fund method is the ideal, impractical one and the Committee's the practical method. With the sinking fund the annual payments with their interest must be kept intact as funds which the stockholders must not profit from, else there will not be sufficient in the fund when the time for replacement comes. Now, in railroad practice, such funds can be used much more profitably in betterments to the property in general, where, of course, they will earn the higher rate assumed as possible by the rest of the property, than if loaned as ordinary investments at interest. This, then, would be the obvious and proper way in which to invest them; but, if so invested, the company must keep their earnings separate, except as used for straight replacements, rigorously withholding all earnings from betterments created by such funds from dividends, else the replacement fund will not be kept inviolate.

It is assumed that Mr. Swain would allow the return from value new as available for dividends, while the sinking-fund accumulation would provide at the proper time for replacements. If these annual sinking-fund allowances are used for additions and betterments, as is shown above to be advisable, can he separate these "fund items", as they might be called, so that the interest allowed on their value new would not be paid as dividends, instead of being held as they should be to build up the replacement fund?

Mr. Snow.

Another objection is the repeated findings of the Supreme Court that an item cannot be held as of the same value as when new after it has passed an appreciable portion of its life term. If we may ignore the Courts, as has been suggested, many things not now thought advisable may be done; but, when these findings appear to be equitable and good law to so many of us, it is a question if it is not best to conform to them with good grace.

The Courts also set forth valid objections to a utility company accumulating a fund of any kind, but hold rather that it should pay as it goes, collecting sufficient revenue to do so.

By the replacement method, as advocated by Mr. Lum, the result is that the customers after the replacement have to pay the bill, rather than those who have enjoyed the use of the item while it was in service. This is seen more plainly in the case of a large item, like a water-works reservoir or a terminal railway station, than in the case of railroad ties, although the principle is theoretically the same.

To show that the method of the Committee in allowing return on remaining value is just and covers the exact new capital put into the property, Table 21 is submitted. Assume a business consisting of ten items of equal value, with life terms of 10 years, and that one item is installed each consecutive year until the ten are in use. The value new is represented by 100, and depreciation is computed by the equal-annual-payment method, recommended by the Committee, on a 7% basis.

The remaining value of each item at the end of each year is shown in the body of the table, and Column 12 shows that after one cycle, *i. e.*, 10 years, the total of the remaining values remains constant at 605.39. Column 14 shows that this sum also represents the total new capital that has been put into the property, assuming that the regular allowances for depreciation each year have been used toward establishing the new item due the following year. Interest on this sum (605.39) plus depreciation, as recommended by the Committee, is manifestly the proper measure of the return from these items of property to use when fixing rates.

If the straight-line method of depreciation had been used, the resulting total remaining value after a life cycle had passed would have been 550.00, and this figure would represent the amount of new capital needed under the conditions named above.

The reason for the smaller capital investment when depreciation allowances are computed by the straight-line method is, of course, that the early payments are large and consequently a greater return accrues to the owner's benefit than happens under the equal-annual-payment method. The final outcome at the end of the life cycle is the same if interest is fully allowed for in both cases.

Mr. Snow.

TABLE 21.—LIFE HISTORY OF A PROPERTY CONSISTING OF TEN EQUAL ITEMS WITH LIFE OF 10 YEARS. ASSUMING THAT ONE ITEM IS ADDED EACH CONSECUTIVE YEAR.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	DEPRECIATED BY THE EQUAL-ANNUAL-PAYMENT METHOD. 7% RATE.												
Years.	Items of Property.										Value new, less depreciation.	Depreciation allowance.	New capital required.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.	No. 9.	No. 10.			
0	100.00	Remaining value.									100.00	.....	100.00
1	92.76	100.00	.....	.....	.....	.....	.....	.....	.....	.....	192.76	7.24	92.76
2	85.02	92.76	100.00	.....	.....	.....	.....	.....	.....	.....	277.78	14.98	85.02
3	76.73	85.02	92.76	100.00	.....	.....	.....	.....	.....	.....	354.51	23.27	76.73
4	67.86	76.73	85.02	92.76	100.00	.....	.....	.....	.....	.....	422.37	32.14	67.86
5	58.38	67.86	76.73	85.02	92.76	100.00	.....	.....	.....	.....	480.75	41.62	58.38
6	48.23	58.38	67.86	76.73	85.02	92.76	100.00	.....	.....	.....	528.98	51.77	48.23
7	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	.....	.....	566.34	62.64	37.36
8	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	.....	592.08	74.26	25.74
9	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	605.39	86.69	13.31
Total new capital in plant.....													605.89
10	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	605.39	100.00	0
11	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	605.39	100.00	0
12	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	605.39	100.00	0
13	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	605.39	100.00	0
14	67.86	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	605.39	100.00	0
15	58.38	67.86	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	605.39	100.00	0

Mr. Snow.

Another objection is the repeated findings of the Supreme Court that an item cannot be held as of the same value as when new after it has passed an appreciable portion of its life term. If we may ignore the Courts, as has been suggested, many things not now thought advisable may be done; but, when these findings appear to be equitable and good law to so many of us, it is a question if it is not best to conform to them with good grace.

The Courts also set forth valid objections to a utility company accumulating a fund of any kind, but hold rather that it should pay as it goes, collecting sufficient revenue to do so.

By the replacement method, as advocated by Mr. Lum, the result is that the customers after the replacement have to pay the bill, rather than those who have enjoyed the use of the item while it was in service. This is seen more plainly in the case of a large item, like a water-works reservoir or a terminal railway station, than in the case of railroad ties, although the principle is theoretically the same.

To show that the method of the Committee in allowing return on remaining value is just and covers the exact new capital put into the property, Table 21 is submitted. Assume a business consisting of ten items of equal value, with life terms of 10 years, and that one item is installed each consecutive year until the ten are in use. The value new is represented by 100, and depreciation is computed by the equal-annual-payment method, recommended by the Committee, on a 7% basis.

The remaining value of each item at the end of each year is shown in the body of the table, and Column 12 shows that after one cycle, *i. e.*, 10 years, the total of the remaining values remains constant at 605.39. Column 14 shows that this sum also represents the total new capital that has been put into the property, assuming that the regular allowances for depreciation each year have been used toward establishing the new item due the following year. Interest on this sum (605.39) plus depreciation, as recommended by the Committee, is manifestly the proper measure of the return from these items of property to use when fixing rates.

If the straight-line method of depreciation had been used, the resulting total remaining value after a life cycle had passed would have been 550.00, and this figure would represent the amount of new capital needed under the conditions named above.

The reason for the smaller capital investment when depreciation allowances are computed by the straight-line method is, of course, that the early payments are large and consequently a greater return accrues to the owner's benefit than happens under the equal-annual-payment method. The final outcome at the end of the life cycle is the same if interest is fully allowed for in both cases.

Mr. Snow.

TABLE 21.—LIFE HISTORY OF A PROPERTY CONSISTING OF TEN EQUAL ITEMS WITH LIFE OF 10 YEARS. ASSUMING THAT ONE ITEM IS ADDED EACH CONSECUTIVE YEAR.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	DEPRECIATED BY THE EQUAL-ANNUAL-PAYMENT METHOD. 7% RATE.												
Years.	Items of Property.										Value new, less depreciation.	Depreciation allowance.	New capital required.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.	No. 9.	No. 10.			
0	100.00	Remaining value.									100.00	.....	100.00
1	92.76	100.00	.....	.....	.....	.....	.....	.....	.....	.....	192.76	7.24	92.76
2	85.02	92.76	100.00	.....	.....	.....	.....	.....	.....	.....	277.78	14.98	85.02
3	76.73	85.02	92.76	100.00	.....	.....	.....	.....	.....	.....	354.51	23.27	76.73
4	67.86	76.73	85.02	92.76	100.00	.....	.....	.....	.....	.....	422.37	32.14	67.86
5	58.38	67.86	76.73	85.02	92.76	100.00	.....	.....	.....	.....	480.75	41.62	58.38
6	48.23	58.38	67.86	76.73	85.02	92.76	100.00	.....	.....	.....	528.98	51.77	48.23
7	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	.....	.....	566.34	62.64	37.36
8	25.74	37.26	48.23	58.38	67.86	76.73	85.02	92.76	100.00	.....	592.08	74.26	25.74
9	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	605.39	86.69	13.31
Total new capital in plant.....													605.39
10	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	605.39	100.00	0
11	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	605.39	100.00	0
12	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	605.39	100.00	0
13	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	605.39	100.00	0
14	67.86	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	605.39	100.00	0
15	58.38	67.86	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	605.39	100.00	0

Mr. Snow.

To show the advantage of the equal-annual-payment method over the straight-line method when rates are to be determined, the diagram and table, Fig. 7, has been prepared. The depreciation by the straight line is, of course, 10 each year. This, added to the interest on the remaining value each year, is shown graphically at the left of the

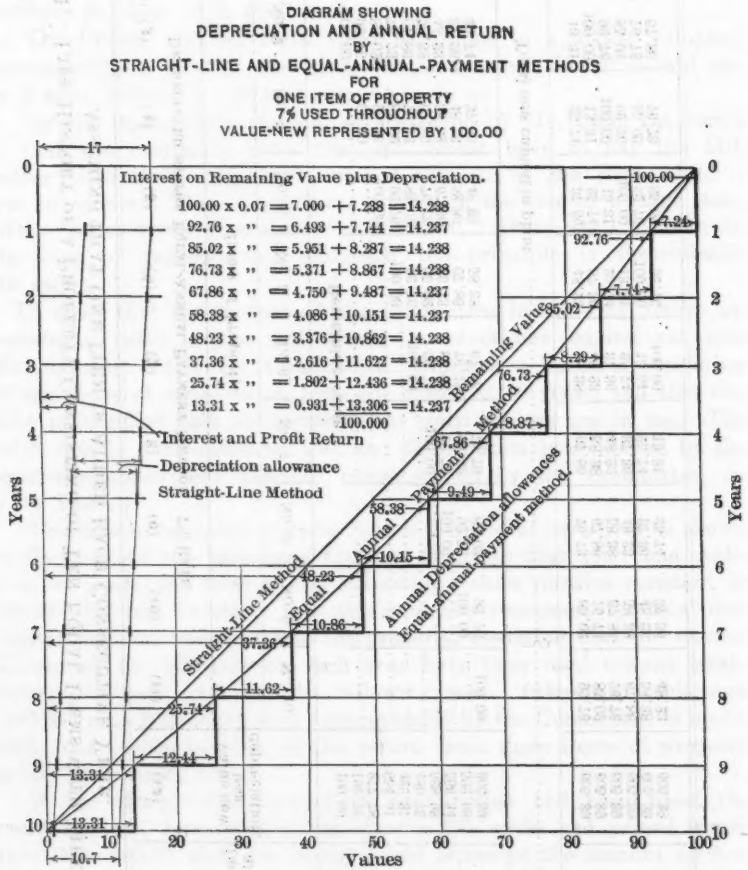

FIG. 7.

diagram. The total return will be seen to vary from 17 at the end of the first year to 10.7 at the end of the cycle, averaging 13.85. The similar features by the other method are shown by the table on Fig. 7, and will be seen to be constant for each year at a value of 14.24. It will be appreciated by all that level rates year by year are to be preferred to those which vary.

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The interest and profit returns on remaining value, varying from 100 the first year to 605.39 at the tenth, by Column 12 of Table 21 is the proper amount to distribute as dividends; and the amounts in Column 13, of course, should be put back into the property. The figures of this column apply to the beginning of the year at which they are placed, or to the end of the preceding year, and, to obtain total return, must be added to the interest and profit return on the capital in the plant during the preceding year. If this is done it will be seen that the rates or gross income should be 14.238 at the end of the first year and direct multiples of this for other years up to the tenth; after which it remains constant at 142.377, or ten times the first return, as it should, as there are then ten items of property involved to one item at the beginning. The sum of Columns 13 and 14 at the beginning of the tenth year is 1 000, equalling ten full items, as it should.

If items of varying value or of irregular periods of installation had been assumed, the calculation would have been more complicated and the result not so clearly shown, but the principle would have been the same, and the equitable division between dividend and depreciation monies as clearly set forth.

If the sinking-fund method is used, the figures of Column 13 will be multiples of the first figure, provided the funds are invested outside, as a regular sinking fund should be. If put back into the property, the column becomes quite complicated, and the proper return available for dividends still more so. Under this last assumption, however, the actual amounts of new capital put in and the yearly returns will be the same as with the equal-annual-payment method shown, but, if the percentage of value new is distributed for dividends, the depreciation funds will suffer.

If, under the sinking-fund method, each item is paid for with new capital and the depreciation fund is invested outside, the new capital at the tenth year will be 1 000, the annual percentage on which is 70, and Column 13 will consist of multiples of 7.2377, which is its first term, the tenth being 72.377, as by the other method. This shows that the difference between these methods is one of bookkeeping, and the objection to the sinking-fund method is that a separate outside fund must be maintained or great risk run of distributing too much in dividends, thereby robbing the depreciation fund.

Considering the replacement method advocated by Mr. Lum, Column 13 would consist of zeros to the end of the tenth year. From the eleventh year onward the figures would be 100. The new capital would increase by 100 each year for 10 years, and the annual returns would be 7 for the first year and multiples thereafter to the tenth year. Hence the public should pay for the ninth year 63, for the tenth year 70, and for the eleventh year 170. This would necessitate

Mr. Snow. a great jump in the rates or a cut in the dividends. For ordinary railroad operation, renewals can be arranged so as to be fairly uniform from year to year, but when a great terminal is rebuilt, the above jolt occurs and, as it is difficult to raise rates, dividends must suffer, as local instances show.

Careful study of the tables in the report and the various ones since submitted by the Committee will show the entire equity of the equal-annual-payment method and its certainty to protect the company owning the property considered.

Mr. Mills. ARTHUR L. MILLS, M. AM. SOC. C. E. (by letter).—When the real estate of a company is of comparatively small value, the treatment of appreciation is of no great importance, as earnings cannot be provided by regulation with any degree of precision; but when the value is large, it may become a serious problem.

The value of the real estate of the four principal railroad lines to Chicago, the New York Central, Pennsylvania, Erie, and Baltimore and Ohio, probably exceeds $1 500 000 000. The appreciation of this real estate has averaged at least 5% per year in the last 20 years. If this appreciation is, in the future, to be considered as part of the income, it means that the estimated income will be greater than the actual income by $75 000 000 per year; whereas the estimated increase in income for all the roads in this territory by the 5% advance in rates which these roads desire, is only about $40 000 000.

The U. S. Supreme Court has decided that the cost of reproduction or present value of the property must include the value of the real estate at the time of the valuation.

The New York Public Service Commission in the First District includes in the probable income the estimated appreciation of the real estate; but the Appellate Division of the Supreme Court of the State of New York, in a decision rendered in May, 1913, refused to adopt this view. The inclination of the Interstate Commerce Commission at that time is indicated in the decision in the Western Rate Case. Commissioner Lane said on this subject:

"Whatever the true economic or legal view may be as to the right of a carrier to consider the increase in value of its land as a part of the value upon which it is entitled to a reasonable return, such increase in value does not of itself establish the right of a carrier to increase rates upon a given service. Certainly if the Supreme Court may decline to lay down the absolute rule that 'in every case failure to produce some profit to those who have invested their money in the building of a road is conclusive that the tariff is unjust and unreasonable', *Regan v. Farmers' Loan & Trust Co.*, 154 U. S., 412, it is a conservative statement of the law to hold that a railroad may not increase the rates upon a number of commodities solely because its real estate has arisen in value."

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"While it is evident, therefore, that each case must be decided upon the facts peculiar to it, the Commission believes it proper in this case to follow the general rule, as stated by Judge Hough of the United States Circuit Court (Consolidated Gas Co. *v.* City of New York, 157 Fed. Rep., 855):

"'Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the one at bar.'

"Thus, land has been taken at its fair value and not at its original cost, and the annual appreciation of land has been treated as a profit. By this method, all property is treated absolutely alike, as Judge Hough suggests. No difference is made, except that as depreciation represents a decrease in assets, it is placed as a *debit* against operation, while appreciation is placed as a *credit*, because it is an increase in assets. Land has sometimes been treated like other property only to a degree; that is, each class has been appraised at its present worth or value. That has been done in this case. But if property has been taken at its *depreciated* value where it has depreciated, an entry must regularly be made in estimated operating expenses equal to the average annual depreciation. Conversely, if land, or any other property which genuinely appreciates in value, is to be taken at its *appreciated* value, then an entry must be made in the estimated receipts equal to the average annual appreciation. Unless this is done, it is obvious that the consumer will be burdened with all the estimated decreases in assets but not credited with the increases in assets. If the principle laid down by the court is to be followed in part, it should be followed in whole.

"It is suggested that the annual increase in the value of land which is treated as income is not actually received. Increase in the value of unoccupied land is not realized until sold or put to use, but it is real, nevertheless, although payment may be deferred. Likewise, payments to the depreciation fund are not actually expended; yet they have been considered legitimate charges in practically every case. Furthermore, the *annual* increment is no more indefinite than the *total* increment—the present value. But if the present value can be determined, it is possible to determine past *annual* appreciation with positive accuracy, for it is only a simple mathematical calculation. It is also probably as easy to estimate increases in the near future as it is to estimate what obsolescence, which is a form of depreciation, there will be in the future.

"Indeed, the problem of handling appreciation is much simpler than depreciation. If the property is growing more valuable, the investor need not worry; and if the state recognizes his right to earn a fair return upon the increase, he is fully protected. It is not necessary that the increase be represented by stocks or bonds, for if the earning power is there, he will receive a return thereon, regardless of the amount of securities. In fact, the existence of an increase which is not represented by securities is an element of safety, a reserve fund of a valuable kind.

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"There is a further similarity. The exact amount of depreciation and the annual rate are not definitely known until the piece of property is actually replaced or has become useless. The total appreciation and the average annual rate are not known until the land is sold, but when it has been disposed of (and plants are continually being removed and the land sold), they become absolute certainties. Why should these matters be considered less definite when applied to land than when applied to the buildings thereon? The depreciation of the buildings is a charge against operation; why should not the appreciation of land be a credit?"

Mr. Lane's statement, "it is a conservative statement of the law to hold that a railroad may not increase the rates on certain commodities solely because its real estate has risen in value", may be fair as an abstract proposition; but it does not follow that an advance in rates should be denied because it would permit a return on the appreciated value of the real estate. The U. S. Supreme Court in many cases has laid down the general principle that the owner is entitled to fair return on the fair value of his property, regardless of cost, provided the rates are reasonable. It would probably be considered against public policy to allow railroads to raise their rates simply for the purpose of paying excessive dividends, even if such an income would be no more than a fair return on the value of their property. It does not follow that the cost of reproduction or present value will necessarily be the controlling factor in rate regulation. The Supreme Court has decided that the original cost, cost of improvements, and the value of the stocks and bonds, must also be considered, and that, regardless of everything, the rates must be reasonable.

Where a railroad needs more money to meet its expenses and also the demands of an increasing tonnage, and to keep up its dividend rate which is not excessive, however, it appears equitable to permit an increase in rates, provided the proposed rates are reasonable and the anticipated income is not more than a fair return on the fair value of the property, regardless of the original cost of the real estate.

Appreciation and depreciation are separate and distinct processes, and each should be considered on its own merits independently of the other and by reason of the effect it has on the present value. It is found necessary to provide from the earnings the cost of the depreciation of the structure, due to wear and tear, etc., for the reason that it is a necessary expense and one that cannot be neglected without damage to the property; but this in itself is not a valid reason for considering appreciation as an income.

Appreciation is not an actual receipt, and is of no assistance to the railroad in meeting its disbursements. To the extent, therefore, that estimated receipts include estimated appreciation, they will, for all practical purposes, be fictitious. Moreover, because real estate has appreciated at a certain rate in a term of years, it does not follow that

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the appreciation will continue at that or any other rate. The value of real estate frequently remains unchanged for a long time and then increases rapidly for a few years. Any estimate of the future appreciation of real estate is speculative, as many purchasers of that class of property know to their sorrow.

Appreciation of real estate is available as income only when the land is sold, and, as a railroad is rarely able to sell its real estate, the stockholders cannot ever expect to realize any income from real estate appreciation.

The real estate of a railroad is different from that of most other corporations. In rare cases only is a road able to dispose of even a part of its land. The demands of a constantly increasing business make it necessary to purchase additional land. It is doubtful if the actual value of the real estate is given any consideration by investors in railway securities. To suppose that the real estate of a railroad may be sold, is to pre-suppose its bankruptcy and abandonment.

Appreciation of real estate differs in character from the depreciation of the physical property due to wear and tear, obsolescence, etc.; it resembles depreciation due to a decline in the prices of rails, equipment, etc., from the prices paid at the time of their purchase, but this kind of depreciation is neglected in a valuation.

Mr. Lane says that unless appreciation is considered a part of the income "it is obvious that the consumer will be burdened with all the estimated decrease in assets but not credited with the increases in assets". The consumer is not burdened with depreciation because it is a decrease in the assets, but because it is a part of the expense of operation. It is something that requires money, and unless appreciation can provide money it does not seem equitable to use it as an offset to depreciation or for any other reason.

One is almost forced to the conclusion that the idea of including appreciation as a part of income did not originate from considerations of equity, but from a desire to find some way to offset the effect of the continuing appreciation of real estate which, in time, will increase the physical values of railroads so that a fair return thereon will permit excessive dividends, and to prevent future reductions of the rates.

It does not seem logical, however, to expect that rates will remain stationary while the cost of handling the traffic is continually increasing. As the volume of traffic increases, it may offset in part the greater cost of labor, supplies, etc., but only in part, because increasing tonnage brings with it the demand for additional facilities.

If the appreciation of real estate is to be considered as a part of the income to which they are entitled, the railroads would be better off if their property did not appreciate in value. For, by considering appreciation as income, they would have to pay from their earnings the

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full amount of this appreciation with the expectation that at some future time they would receive the interest thereon; but there is no certainty that they will receive this interest, for, at the time of the next adjustment of rates, the physical valuation might be neglected, other considerations being the controlling factors.

If part of the return to which a road is entitled is represented by the non-tangible appreciation of the real estate, there will be, theoretically at least, a deficit in money with which to meet disbursements. It has been suggested that securities can be issued to make up this deficit; but this, in effect, would be to pay operating expenses or dividends in part from the sale of stocks and bonds. It would hardly be equitable to ask a stockholder to waive his dividends wholly or in part because the real estate of the company was increasing in value, knowing that probably it will never be of any benefit to him in the matter of dividends or in enhancing the value of his stock.

The appreciation of real estate is no more a part of the fair return to which a railroad is entitled than losses incident to the enterprise are a part of the expenses on which rates are to be based. If the consumer does not guarantee losses, he is not entitled to the profits. Suppose it was found that the real estate of a railroad was depreciating, would it be considered fair to call this depreciation a part of the expenses to be provided for from earnings? Such a proposal would be decided negatively, for the reason that changes in value are a risk in his enterprise which the investor assumed.

Again, suppose there are two roads, similar in value, except that one owns its real estate and the other leases it at a rental based on its value, provision being made for revaluation. Then, if appreciation is considered as income, the second road will theoretically be allowed the larger rate of income from earnings, for the reason that the expenses of the second road will include the rentals, while the income the first road might be allowed on the value of its real estate will be offset by the appreciation.

As an illustration, take two railroads, one valued at $100 000 000, of which $30 000 000 represents the value of the real estate which has appreciated at the rate of 5% per year. The other road owns no real estate, and is valued at $70 000 000. It uses real estate valued at $30 000 000, paying therefor as rental 6% on its value. It is assumed that each company will be allowed to earn 6% on its valuation. The first road will be allowed to earn $6 000 000, but in this income will be included $1 500 000, which has been the yearly appreciation on its real estate, leaving a balance of cash available for interest and dividends of $4 500 000, or 4½% on its value. The second road will include the rentals in its expenses and will be allowed to earn 6% on its value.

Theoretically, then, if the appreciation of real estate is considered as income, the road which owns its real estate is at a disadvantage as

compared with one that has no such investment, but pays a rental for its use. The word, theoretically, is used because regulation of railroad earnings with any degree of precision by changes in rates is impracticable. Railroad rate regulation is a more complicated problem than in the case of public service corporations with no competition and with service limited to a comparatively small area. Mr. Mills.

To consider the appreciation of real estate as income is confiscatory in character, for the reason that it deprives the owner from any benefit from the future appreciation of the real estate. If he pays for it yearly out of the "fair return" on the chance of getting interest on it some time in the future, the appreciation of his real estate has been practically confiscated.

Briefly, then, for the following reasons, appreciation of real estate should not be considered as income:

It is indeterminate and speculative;
It is not a tangible receipt;
It is different in character from the depreciation of the physical structure, due to wear and tear, for it is a matter of price which may advance or decline.

Such a practice would have the effect of giving the consumer the benefit of the profits due to appreciation, without requiring him to contribute to the losses of a similar character.

It would create a deficit, theoretically at least, which could be overcome only by the sale of stocks or bonds, which, in effect, would be to pay expenses or dividends in part from the sale of securities.

It is confiscatory, because whenever a property is sold the owner is entitled to its full value, whereas if he is required to pay for the appreciation of his real estate, he is deprived of a part thereof.

J. M. SCHREIBER, M. AM. SOC. C. E.—The speaker is particularly interested in the section of the Committee's report relating to depreciation, because he has been identified with similar work undertaken by the American Electric Railway Association. About three years ago, that Association, appreciating the importance of depreciation, through its allied associations, the American Electric Railway Engineering Association and the American Electric Railway Accountants Association, appointed a joint committee of six members, comprising three accountants and three engineers, to investigate the subject. This Committee is still in existence, and is known as the "Joint Committee on Life of Railway Physical Property". The subject was taken up with serious thought and energy, and sincere and earnest endeavors were made, in order to comply with its purpose. The data, secured from the properties with which the individual members were connected, were not only examined, but the Committee got in touch with qualified representatives of almost all the large electric railway companies in the Mr. Schreiber.

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United States. Amplified inquiry forms were sent out and personal solicitations were made, in an attempt to obtain figures expressing the life of railway physical property, or definite tables which would accomplish the same results.

In general, it was found that those having practical experience, and, in many instances, the responsibility of a particular item of railway property under investigation, felt that any attempt at present to express the life of railway property explicitly, was at best a guess.

After three years of study of the life of railway physical property, from every source obtainable, including the accumulation of a more or less complete bibliography on the subject, this Joint Committee reached the following conclusions:

(1) That the basic elements which determine the life of railway physical property which affect its usefulness are:

(*a*) Use,
(*b*) Climatic and soil conditions,
(*c*) Maintenance,
(*d*) Inadequacy,
(*e*) Obsolescence,
(*f*) The human element,
(*g*) The public demand, and
(*h*) Earnings.

(2) That it is not practical, or even theoretically possible, to assemble compositely these several elements into any form of logical "Life Table" that will apply equally in Maine and in California, in Minnesota and in Louisiana, or on different routes or lines of the same system embracing such physical property.

(3) That the ultimate solution of depreciation of railway physical property is insured earnings.*

It has happened that a great many of the points covered in the conclusions cited have practically been acquiesed in by those who have discussed the subject. It seems that the only way to consider depreciation, if at all, is as suggested by Professor Swain—to designate or set apart a certain portion of the earnings each year to be used for that purpose. Depreciation is an operating problem, and is simply deferred maintenance, and generally, if the amount is proper, allowing for reasonable contingencies, it is used up as operation proceeds year by year. This is because the railway physical property does not have to be replaced all at one time, but on a piecemeal basis, as it were.

Now, as to the important subject of proper general methods for valuation for the purpose of rate-making, it is the speaker's firm opinion that there is only one consistent method to follow, and that is to

* A full report of this Committee, with discussions, may be found in the *Proceedings*, Am. Electric Ry. Assoc., for 1912 and 1913.

ascertain the cost of reproduction new; and, that depreciation should not be taken into account. All the user is concerned about is proper service, and he does not care a whit about depreciation, if the service is not impaired. The speaker desires to call attention to a recent decision of one of the commissions, which is pertinent in connection with the report of the Special Committee on Valuation. It is contained in the Sixth Annual Report of the Public Service Commission of Montana for 1912 and 1913, in the case of J. F. Edwards *vs.* The Helena Light and Railway Company, in which the Commission says: Mr. Schreiber.

"Assuming that rates were being made for a new plant, it would be the total capital that must be considered as entitled to bear interest, as there would be no accrued depreciation. Depreciation is a liability against the property, which must be accounted for, but in making allowance for future depreciation, it is not the intention to provide for accrued depreciation, which it is assumed, has been taken care of. To simplify the matter, let us assume that an investment is made in 1903 of $100,000 under a twenty-year franchise, rate of interest allowable 10% per annum, and figuring 5% per annum depreciation. At the end of ten years, or in 1913, the property will have depreciated $50,000 and has a remaining value of a like amount. Then, if rates are made, based on the depreciated value, they must be one-half of the original rates, although the service may be just as efficient as it ever was, and in ten years more, the physical value of the plant would be nil, and likewise upon the same basis of reasoning, the utility would not be permitted to charge anything."

In conclusion, the speaker appreciates thoroughly the great task undertaken by the Committee on Valuation for the Purpose of Rate-Making, and thinks it deserves a great deal of credit. He agrees with a considerable portion of the report. As a member of the committee of the American Electric Railway Association, he is quite sure that it will be very glad to co-operate in every way with the Valuation Committee, and furnish any information and data on the particular subject of depreciation at its disposal. The speaker would also suggest that the Committee confer with the National Electric Light Association, which no doubt would also be willing to furnish pertinent data on the subject.

Certainly any conclusions on the whole report should not be accepted until the utilities, which are so vitally interested in the question, could at least have an opportunity to present their views properly.

J. P. NEWELL,* M. AM. SOC. C. E. (by letter).—Should depreciation be taken into account in fixing rates? If by this is meant, "Should rates be reduced as the depreciation increases?", the question must be answered in the negative. Assuming that the service rendered remains the same in quantity and quality, the rates should remain the same. Mr. Newell.

* This discussion was presented before the Portland, Oregon, Association of Members of the American Society of Civil Engineers, at its meeting of April 4th, 1914.

Mr. Newell.

Any other policy certainly would not be attractive to capital, nor would it be of advantage to the public. As the total return during the life of the plant must be the same in any case, diminishing rates would require a higher charge in the beginning. Still, depreciation must be taken into account for the benefit of the company, not of the public, for depreciation is not a penalty assessed against the company for the deterioration of its plant; it is an amount added to the rates to repay the company for this same deterioration.

Much of the confusion of thought concerning depreciation arises from failure to distinguish between the standpoint of the public and that of the management. The prudent manager of a public service corporation will open a depreciation account, and will take good care that the amounts credited to it do not pass to the stockholders in dividends, leaving him unable to provide for renewals and replacements when needed. That, however, is of no interest whatever to the public. Fix rates that will insure capital a fair return, and renewals are certain to be taken care of. Even if the existing company dissipates its capital and is unable to replace it, some new company, knowing that a fair profit is assured, will be found ready to step in with new capital and take up the work.

The rate-regulating power need concern itself with only one question: At what figure must rates be fixed so that, at the end of the life of the plant, the owners will have received the full amount of the operating expenses, the value of the plant, and a fair return on their investment?

One method of finding the answer to this question is that used in dealing with a peddler: Find out the least he will take and give it to him. If we can do this, we need only divide the total amount by the assumed life of the plant, and we have at once the annual charge for service. The capitalist may put the receipts in his pocket, divide them up in his accounts in any way which suits him, and spend them as he pleases; and it is in accordance with this principle that the fairness of all rates must ultimately be judged. The least that capital will take, under conditions such that all capitalists are free to invest, is what it should be allowed to charge.

It would take years, in many cases decades, however, to apply this method. Besides, we must deal with many utilities, built under different conditions, the owners of which are not free to act on the above principle. We must deal justly with these men, who are, in a sense, at our mercy, or later we shall find ourselves at the mercy of other capitalists who will refuse to invest in other utilities which we need.

Some contend that rates should be based on the reproduction cost new and not on the depreciated value. True, as the rates must be sufficient to return to the owner an amount equal to this cost. The owner's profit, however, cannot justly be based on this reproduction

cost new, if part of it is returned to him annually through the depreciation allowance. Mr. Newell.

Compare the case with that of a loan. Suppose B borrows $100 from A at 6% interest, the principal to be repaid in 10 years. Anticipating B's inability to pay the whole amount at one time, A insists that B, in addition to the interest, shall make an annual payment on the principal. At the end of the first year B will then pay $10 plus interest on $100, or $16. At the end of the second year his payment will be $10, plus interest on $90, and so on to the last year when he will pay the last $10 of the principal and interest on $10.

Certainly no one will contend that A should continue to receive interest on the original $100 for the entire time, on the ground that he expects to make B a new loan at the expiration of that time. Neither should an investor expect to receive a profit on capital which has been repaid to him. Nor does this depend on the "adequacy" of the depreciation allowance. If that allowance be small, only a small part of the investment is wiped out, if large, there is a larger reduction in the capital; but the principle is the same in either case. If at any time it is found that the life of the plant has been assumed at too high a figure, and the rate of depreciation is correspondingly too low, the remedy is to increase the depreciation allowance, and, consequently, the rates charged the public, and to make this increase sufficient to cover the error for past as well as for future years. Care should be taken to assume depreciation high enough in the first place so that adjustments will reduce rather than increase rates. In fixing the rates of an existing plant, if it can be shown that its income has been too small in the past to produce a fair return, or that the profits have been excessive, the rates should be governed accordingly. If neither of these conditions can be satisfactorily proved, however, we must proceed on the assumption that the plant has been earning a fair return and no more.

Consider the case of a plant fully equipped, with business neither increasing nor decreasing, so that no extensions are required; the cost of reproduction new and the expected life of the plant have been ascertained, and the fairness of the existing rate is to be determined. The rate under consideration produces a gross income which is constant from year to year. The operating expenses are assumed to be constant, so the sum of the depreciation and the profit, or return on capital, the other two elements composing the gross income, is also constant.

If we adopt the "Straight-Line" Method of calculating depreciation, both these factors become constants, also; that is, we have a constant return based on a constantly diminishing capital. There is no inherent injustice in this to either party, but the rate of return on the existing investment must be too low at first, and too high at the last. This is illogical and unnecessary. It opens the way for complaint from the

Mr. Newell.

company in the first period and from the public in the last. It adds also to the difficulty of adjustment, if the assumed rate of depreciation is found to be erroneous.

Let us then assume that the rate of return on the capital remaining in the plant must be the same each year. We now have these conditions: The gross income, a constant, is made up of three parts:

(1) Operating expenses, also a constant;
(2) Depreciation;
(3) Profit.

The sum of (2) and (3) is constant. The rate of profit or return is constant, but, being based on a valuation which diminishes year by year, the actual return also diminishes, and the depreciation correspondingly increases. The value and the life of the plant being known and the sum of the depreciation and the profit being fixed, as stated, it becomes a matter of mathematical calculation only, to assign the proper amount to each. The depreciation must be fixed at such an amount, for the first year, that the depreciation for each year plus the return on capital, based on the depreciated value for that year, shall equal the given sum, and that the sum of all the annual depreciations during the life of the plant shall equal the value of the plant. Solving, we get the rate of return on capital produced by the given rate, and the amount of capital repaid each year.

In the illustration already given of a loan, B pays $16 the first year and $10.60 the last. If, instead, he makes a payment of $13.59 each year, from which the interest due is first deducted, and the balance applied on the principal, he will likewise extinguish the debt in the given time. In the first year his payment on interest will be $6 and on principal $7.59; in the tenth year, the interest will be $0.77 and the principal, $12.82. Similarly, the annual repayment of capital from a uniform rate should increase as the depreciated capital diminishes. There should be no attempt to make the depreciated allowance conform to the actual visible depreciation from year to year. Fluctuations in the observed conditions should be taken account of only as they affect the life of the plant.

The results obtained by this method correspond to those given by the tables accompanying the report of the Special Committee. The Committee, unfortunately, has confused the matter somewhat by presenting a table in which one percentage is used in calculating depreciation, and another for profit. The sum of the results thus obtained not being constant, it follows that rates cannot be constant.

A concrete example will make these principles clearer. Assume a plant costing $100 000, with a life of 10 years, and operating expenses of $6 000 per year, and fix the fair rate of return on capital for this

kind of investment at 7 per cent. This gives all the information needed for the establishment of rates. Mr. Newell.

By the "Full-Value" Method, the annual depreciation will be $10 000, the return on capital $7 000, and the rates $23 000.

The totals for the life of the plant will be:

Operating expenses.....	$60 000
Depreciation	100 000
Profits	70 000
Total	$230 000

It will be seen from Tables 22 and 23 that the depreciated value of the plant at any given time has no bearing on the rates. The whole matter of depreciation is considered, once for all, when the length of time is determined during which the value of the plant must be repaid out of the rates. Any further evidence of depreciation is important only as indicating error in fixing that time.

In all the previous discussion, the effect of renewals and replacements has been ignored; but these do not affect the principles stated. The item replaced has been depreciated to zero condition, and then renewed to 100% by the introduction of new capital. The life of the plant is a composite of the lives of all the parts, in proportion to their respective values.

Fair rates may also be determined by the Sinking-Fund Method, provided the profit is based on the reproduction cost new, and the interest on the sinking fund is calculated at the same rate as the profit. Under this method, instead of handing to the owner each year certain varying amounts, the sum of which will equal the value of the plant within its life, and which are to be deducted each year from the capital on which a profit is permitted to be earned, a uniform amount, less than the average of those above, is assumed to be set aside each year. These uniform amounts, at compound interest, will equal the value of the plant at the end of its life, but they will not do so unless the owner continues to apply his industry and skill to them. As the capital is gradually transferred from plant to sinking fund, but remains always engaged, the fair profit must be based on the full, and not on the depreciated, capital. This is true, no matter by what means the owner chooses to make the sinking fund earn the required interest.

It is commonly assumed that interest on a sinking fund should be calculated at a very low rate, such as money can earn with no risk and with the minimum of attention. To the writer this appears clearly erroneous. The owner is paid not only for the use of his money, but also for his skill, industry, intelligence, and risk. These factors he is paid for applying to the entire capital, whether it be all in plant,

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TABLE 22.—By "Straight-Line Depreciation."

Year.	1	2	3	4	5	6	7	8	9	10
Operating expense	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000
Depreciation	10 000	10 000	10 000	10 000	10 000	10 000	10 000	10 000	10 000	10 000
Profit	4 238	4 238	4 238	4 238	4 238	4 238	4 238	4 238	4 238	4 238
Depreciation value	100 000	90 000	80 000	70 000	60 000	50 000	40 000	30 000	20 000	10 000
Rate of profit	4.24%	4.72%	5.30%	6.05%	7.06%	8.47%	10.60%	14.18%	21.19%	42.38%

Annual rates	$20 238	
Total operating expense		$60 000
" depreciation		100 000
" profits		42 380
		$202 380

TABLE 23.—By "Equal-Annual-Payments."

Year.	1	2	3	4	5	6	7	8	9	10
Operating expense	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000
Depreciation	7 238	7 744	8 287	8 867	9 487	10 151	10 862	11 622	12 436	13 306
Profit	7 000	6 494	5 951	5 371	4 751	4 087	3 376	2 616	1 802	932
Depreciation value	100 000	92 762	85 018	76 731	67 865	58 378	48 226	37 364	25 742	13 306
Rate of profit	7%	7%	7%	7%	7%	7%	7%	7%	7%	7%

Annual rates	$20 238	
Total operating expenses		$60 000
" depreciation		100 000
" profits		42 380
		$202 380

or partly in sinking fund. As he is paid, through the rates, for such management as will cause the sinking fund to earn the given rate, he cannot well claim that it is actually earning less. Mr. Newell.

The best possible proof of the correctness of these two methods of calculating fair rates is to be found in the fact that, though differing widely in principle, they give identical results.

If the principles governing depreciation have been correctly stated, it follows that as far as rate-making is concerned, the public service corporation need concern itself about "depreciated condition" not at all, but about the rate of depreciation a great deal. Proof of a short life for the plant will do more than anything else to justify high rates. Such proof will likewise establish a low depreciated value, which may affect the company's interests in the way of sale or taxation values, or borrowing power, but not in respect to rates.

The appraiser, instead of constructing mortality tables by which to determine the condition percentage, should observe conditions in order to make up mortality tables. Every item of the appraisal should have given its age and expected life, as well as the reproduction cost. Depreciation can be expressed only by the ratio of remaining to total life, as depreciated value is a function of the rate of return on capital as well as of length of life. The composite length of life of the plant must then be determined by calculation, the operating expenses by the company's accounts, and the fair rate of return by the judgment of the public service commission. From these three factors and the reproduction cost new, rates properly to be charged the public can be calculated by either of the methods discussed.

The writer prefers the Equal-Annual-Payment Method, because it takes account of all the facts all the time. It gives a "present value" which is also the true value for purposes of sale and taxation. As it reckons profit on depreciated capital, it is more nearly in accordance with Court decisions, and is more likely to meet with popular approval.

GARDNER S. WILLIAMS, M. AM. SOC. C. E.—On its appointment, this Committee was designated a "Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities." The speaker regrets that thus far the Committee does not seem to have risen to the plane required for such a consideration of the problem submitted, as its importance demands, and as some, at least, of those who stood sponsor for the appointment had reason to anticipate. Mr. Williams.

In the speaker's understanding, the Committee was "to formulate principles," not to accept rulings already formulated under the exigencies of particular conditions, especially as in the majority of cases those rulings have been formulated by persons far less qualified to

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deal with the subject, in its entirety, than it is proper to assume the membership of this Committee to be.

Further, it is the belief of the speaker that neither the Committee, nor any one else, should accept dicta as decision, nor opinion as law; or even accept decision as final. For 300 years the Courts steadfastly refused to admit opinion evidence, and it is within the memory of some of the members of this Society when it was absolutely excluded; but where in America to-day is the higher Court that does not receive and consider it?

To this Committee, in the discharge of its prescribed duty, as the speaker understands it, the existing interpretations of the law and the wording of special acts are of no consequence, until the Committee, after careful and mature deliberation, is prepared to say that those interpretations and the wording of those acts are just, equitable, and fair to all concerned in all cases. If the work of the Committee were to have been bounded and controlled by legal decisions and legislative enactments, it should have been empowered to employ those expert in the law to perform its work, or the whole question should have been referred to a bar association. If, in dealing with this subject, the American Society of Civil Engineers cannot get back to, and enunciate, those fundamental principles of rights and justice that underlie and overrule all man-made law, then, in the speaker's opinion, again, this Society should let this subject alone.

In importance, the question referred to this Committee overshadows every other question this Profession is now or has ever been called on to consider. Of the estimated fixed wealth of the United States, exclusive of real estate, nearly 60% was invested, in 1904, in properties within the purview of this Committee, and the investment therein has increased 50% in the last 10 years. On all hands the rumblings of a coming strife between the masses and the classes are to be heard. State socialism is being advanced as a panacea for the ills bred in ignorance and nurtured in indifference, and engineers, trained by the responsibilities of their profession, are the one class fitted to rise between the contending forces and present a correct and safe plan of future procedure. To no class other than themselves does the advance of socialism presage such disaster. Socialism stands for the mediocre man, for the ignoring of special ability, and the suppression of more than average energy. This Profession exists as the foremost exponent of special ability and increased efficiency. The two ideas are absolutely inconsistent and incompatible, and the engineer must recognize the fact and enter the combat accordingly. He must qualify himself to explain clearly to his fellow-citizens the true relations between capital and labor, and the rights and privileges of each, and must take the position, in the community, as a leader

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of its thought, for which his training in exact reasoning should have prepared him.

The valuation of the property of public utilities may be undertaken for any one of five different purposes, which, in the order of their justification, may be stated as follows:

A.—For the purpose of forced acquisition;
B.—For the purpose of sale or purchase;
C.—For the purpose of taxation;
D.—For the purpose of issuing or limiting the issuance of securities;
E.—For the purpose of establishing rates.

(*A*).—In valuation for the purpose of forced acquisition, or, as it is commonly called, condemnation, the provisions of our National and State Constitutions set forth the principles to apply, and no one questions the privilege of sovereignty to exercise the right of eminent domain when such exercise is found to be for the general welfare. The long-established rule that the owner is entitled to the fair value of his property, unaffected by past or proposed legislation to the benefit of the purchaser, has been interpreted to mean that he is entitled to receive the price on which a willing purchaser and a willing seller would agree, for the property to be conveyed. It may be justly questioned whether this is in all respects a fair treatment of the owner, who may not be a willing seller, preferring to any other the particular industry in question as a matter either of choice or of sentiment. Whoever does not recognize the force of sentiment is unfit for citizenship in either a democracy or a republic. Charles Kingsley, as an Englishman, expressed a doubt of its existence as applied to inanimate concepts, and Edward Everett Hale gave an American's answer in "The Man Without a Country." To him, bereft of sentimental association by the appropriation of the result of the labors of perhaps a lifetime, compensation for sentiment is due, and must justly become an item of value recognized in the exercise of the right of eminent domain.

(*B*).—In valuation for the purpose of sale or purchase there again can be no question of the right of procedure, and, as both parties are interested in obtaining the same information, the rules of accounting are to be followed naturally, and the result arrived at should be the price on which the willing seller and the willing buyer can agree.

(*C*).—In valuation for the purpose of taxation the result is influenced by other conditions than in the two former cases. For many years it has been a principle of taxation in America that taxes should be levied only on physical properties, and it is as yet in only some of the States that a deviation from this principle exists. If it

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were not for this restriction, the value for the purpose of taxation should be the same as for purchase or sale.

(*D*).—In valuation for the purpose of issuing or limiting the issuance of securities, there is first encountered a question of right to value. So far as the owner's inquiry into the value of his property for his own information, or for that of the proposing purchaser of its securities, is concerned, there can be no question; as to the right of sovereignty to step in and demand or pass on such a valuation, there may be grave question. Waiving for the time the consideration of such right, however, there appears to be no reason for this valuation to be different from that for purchase and sale. Indeed, the two cases are similar in all respects, for the bondholder may be conceived of as buying the plant in whole or in part, and as allowing to the owner for its operation and maintenance so much of its income as exceeds a certain stipulated sum.

(*E*).—Finally, the right to a valuation for rate-making by the State, it must be admitted, is only to be derived by an extension of the powers of Government never contemplated by the founders of our commonwealth, and if by the term "investment" the Committee means financial investments, the speaker most emphatically takes issue with the statement on page 11 of the report that "it would be entirely just and equitable, and it is highly desirable to provide by law that future public service properties should be valued on the basis of their actual reasonable existing investment, and to determine or limit rates upon such a valuation if the service rendered permits," and he further emphatically denies that there is either justice or equity, or even common sense or practicability, in the proposition therein advanced.

As an example of the inadequacy of cost to determine value, consider the case of a horse about to enter a race for which a purse of several thousand dollars is offered, and that his ability to win the race is assured. Would any one say the value of that horse at that time was represented by the cost of siring, foaling, raising, and training him, plus, if you will, an allowance for deterioration of his dam?

Following the suggestion of the Committee to its logical conclusion, it results that the illy devised, illy designed, expensively constructed, and extravagantly operated plant is put on a par in earning capacity with one prudently devised, well designed, carefully constructed, and economically operated. In other words, the Committee omits that greatest element of value, in all business enterprise, that element without which there could be no variation in value in the same or similar situations, that element which lies at the foundation of all progress, brains. Of all persons on earth, the engineer should be the last to overlook brains. Brains, and brains only, differentiate

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him from the laborer in the street, and our patent laws recognize the compensation due to brains.

Take the first entry of brains into a proposition, its conception; if the idea be novel and the business developed on it be successful, then the idea was valuable and is a part of the investment. Let it be a case of a water power; some one conceives the development, some one works out the general scheme and, for this service, usually receives or retains a portion of the corporate stock for which he makes no other payment. In the search for tangible property, behind the investment, this block of stock will be termed "water", but it is really "brains", and is just as much entitled to recognition as a factor in the development as the money actually paid to labor for the construction of the work; but where, beyond the value of the time spent, is there recompense allowed for this item in the scheme of the report?

Again, a corporation secures the services of an engineer who is an expert, and gives him an interest in the property in addition to the fee for his services. He produces for them a plant capable of converting the power of Nature to the use and convenience of man with a higher degree of efficiency than has been accomplished in any previous plant, and besides doing so, he reduces the construction cost per unit of capacity. Are not the brains of that engineer as put into that plant entitled to be counted as part of the investment in that plant, and is it not right and proper that this plant economically constructed and economically operated should be entitled to earn a higher rate of interest than the plant thrown together haphazard, wasting perhaps 50% of the natural energy it is purposed to convert?

These are some of the questions to which one looks in vain for an answer in the Committee's report, and they are the questions for which a formulation of correct principles of valuation must provide.

The programme outlined in the report, and in every discussion of the subject with which the speaker is familiar, puts a premium on bad judgment, bad design, bad construction, and bad management, and a penalty on their opposites. To such a programme neither the engineer nor the public can afford to subscribe.

If a property be only worth its cost, what is the inducement to produce it? Take, as an example, the land needed for a reservoir site. A part is secured at the price of ordinary farm land and the last few parcels at three or four times that price. When all that land has been brought together into one complete whole, is it worth no more than when scattered in isolated parcels under diverse ownership? Is it worth no more as such a whole than the cost of its acquisition?

Is a watch ready to be wound and to run only worth the cost of its component parts and the time of the mechanician who puts them together?

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The purpose of the public service corporation is to serve. The value of a service has only this connection with its cost, that it must not be less than the cost or the service cannot be rendered, and the compensation to be demanded must not be greater than the value, else no one will accept the service.

To determine properly the rate for a service, its value to the recipient must be established, just as the value of any other article or concept to be marketed must be established.

This is a broad question, a hard question, and it involves the same problems as those confronting the investor every day, that this business may be highly lucrative, while that may be a losing venture, when, in either case, the worth of the physical property has no determining influence on the value of the project.

There are in a certain locality two water-power plants designed to serve the same community. One cost about $300 per horse-power of machine capacity and the other less than $100, on the same basis. In one of them the advantages of expert training and brains were utilized to the utmost, and the plant has proved a model for numerous later ones scattered throughout the country. In the other the brains that entered were used mainly in copying the mistakes of previous designers and in building up a huge physical investment on which bonds could be sold. The latter concern is bankrupt, the former is paying excellent dividends. If the rules laid down by the Committee be accepted, the bankrupt concern would be earning a mediocre return on a bad investment, and the other would have its earnings reduced one-half, while the customers of the one plant would pay three times as much as those of the other for the same service. In its work the Committee, like many of our public service commissions, seems to have started from the wrong premises. Even in dealing with the value of physical property alone, which, by the way, the Courts have usually held to be only an indication of value, there is apparent a misunderstanding of depreciation. The substance of the definition usually accepted for this term is the value of the service already rendered by an article not new. In a valuation, this has nothing to do with the case. So far as its value is concerned, whether the property be a railroad track or a jack-knife, its past service is of no interest; it is its future possibility of service that is involved. The true definition of depreciation as used in valuations then is as follows:

"Depreciation is the difference between the present worths of future service and scrap value of a new article, and of the article under consideration."

If this definition be gotten clearly in mind, it will help in wiping out some of the difficulties contained in the subject of valuation.

GEORGE F. SWAIN, PAST-PRESIDENT, AM. SOC. C. E.—The speaker would say at the outset that he has read this report with greatest interest, and considers it, in regard to everything except the matter of depreciation, the most sane, sensible, and logical discussion on the subject that he has seen. It is clear, forcible, fair, and accurate. It presents the various points impartially, at the same time giving the conclusions which the Committee deems justifiable, and with which, in most cases, the speaker cordially agrees. It brings together a great number of facts and illustrations drawn from experience which have never been collected before, and which cannot fail to be of great value in the future consideration of the subject. The thanks of the Society are cordially due to the Committee for its work. Mr. Swain.

With reference to the question of depreciation, however, the speaker believes that the views of the Committee are lacking in clearness, that they involve confusion of thought, and are misleading. He, therefore, wishes to give his own views on the subject, though he fully realizes that he is quite as likely to err as the members of the Committee. This subject of valuation is one which is in the making, and it must be expected that opinions, both of individuals and of the Courts, will experience modifications as the subject becomes more definitely understood and the principles definitely established.

The question of depreciation is twofold:

First.—What is the actual depreciation?

Second.—What value of the property shall be taken for the purpose in view?

With reference to the actual depreciation, this, as every one recognizes, is not a matter of computation or theory, so much as of actual inspection. The actual depreciation will depend on the maintenance. If a steel bridge has been kept well painted and in good repair, it may not have depreciated at all; that is to say, it may be exactly as good as new, although, on account of obsolescence, or, in this case, the fact that increasing weight of rolling stock may require a new structure before the old one is worn out, it may be less suitable for present purposes than when built.

Actual depreciation, therefore, depends on maintenance, and is to be determined largely by inspection.

With reference to the value of the property which is to be taken, the speaker is glad to know that the Committee definitely states that this value depends on the purpose in view, and may differ according to that purpose. It is true, of course, that a public service corporation would generally be willing to have the same valuation used for taxation as for rate-making, provided a fair value for rate-making purposes is allowed, because the rates should be adjusted so that a fair return will be allowed to the owners after taxes have been paid. This being

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the case, it is evident that it can make no difference to the company how high the taxes are, if such fair return on a fair value is allowed. So far as physical property is concerned, however, a valuation for taxation would not necessarily be the same as for rates. One instance alone will illustrate this point. The construction of a railroad station may have required the taking of a piece of city property with the buildings on it, the latter having been afterward destroyed. A fair valuation for rate-making would include, of course, the cost of the buildings, but these having been destroyed, there is no taxable value to represent their cost except an intangible value. If, therefore, the same value is used for rate-making as for taxation, then, so far as taxation is concerned, this value is not that of the physical property alone, but includes an intangible element.

Coming now to the main point which the speaker wishes to discuss, the burning question with regard to the value which is to be taken as the physical value for purposes of rate-making, to which the report of the Committee is limited, is the question whether a fair return should be allowed on the cost-of-reproduction-new, or on the cost-of-reproduction-new-less-depreciation. With regard to this, the report of the Committee states definitely in several places that the cost-of-reproduction-new-less-depreciation should be used. For instance, on page 10, it is stated:

"The result obtained after making deductions for obsolescence would be the value of existing items of property new, which should be further diminished by the net amount of the depreciation of the existing property."

Again, on page 19, the report states:

"In order to complete the valuation by this method [reproduction-new], it is necessary to make a deduction from the values obtained as above stated, for the net accrued depreciation of the physical property due to age and other causes."

Once more, on page 32, it states:

"A depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

Again, on page 72, the distinct statement is made: "The depreciation value of property should be basis for rate-making."

However, in the Addendum to the report, page 3, it is stated:

"That there are certain methods of compensating for the loss of value due to the depreciation of perishable property, which require that the property be valued as if new in order that injustice may not be done to the owner of the property, but that there are other methods

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of compensating which require that the property be valued at less than its value new, or, in other words, at a depreciated value, in order that injustice may not be done to the public."

In other words, the Committee definitely states here that whether the cost-of-reproduction-new or cost-of-reproduction-new-less-depreciation shall be taken depends on the method of depreciation allowed.

In the speaker's opinion, the method of depreciation to be allowed is an accounting problem, the object being simply to set aside out of earnings each year such sum as may enable all renewals in kind to be made without any increase of capital.

The speaker contends that a depreciation allowance may be considered, not, as the Committee states, as "a payment from the rate-payer to the corporation of a part of its investment", but that it is a payment of the same kind as one for maintenance. It is a payment to ensure the permanent maintenance of the plant in kind without increase of capital. Moreover, it is a payment to the corporation and not to the stockholder who has invested the money. It is a payment which should not reach the stockholder, but should be held by the corporation for a specific purpose, *viz.*, the indefinite perpetuation of the property.

In general, the proper method of allowing for depreciation appears to be clearly the sinking-fund method, because that is most favorable to the public and equally favorable to the corporation.

The Committee is perfectly right in stating (Addendum, page 3): "that the owner of a public utility property is entitled to have the rates made high enough, under normal conditions, to pay for the lessening value of the perishable property, so that when such property goes out of use he shall have received, not only a fair return for the use of the capital invested in such property, but shall also have the capital invested in it paid back to him," except that the original capital should not be paid back to the investor, but should be held by the corporation for renewal when the time comes.

In other words, capital honestly and properly invested should not only receive permanently a fair return, but the rate should be high enough to return to the company an additional amount each year, which, if safely invested, will amount, when the property requires renewal, to the cost of renewing it in kind, that is, as it was originally, without any additions or betterments.

The sinking-fund method, as previously stated, provides for this with the least payment from the rate-payers. The latter pay each year, not only enough to give a fair return on the capital, but a sum which, if placed at interest, will amount to the cost of renewal when renewal is necessary. These payments for depreciation by the rate-payer should be allowed to earn interest, and if so allowed, the yearly payments will

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be smaller than if not allowed; the rate-payers are just as much entitled to claim that interest should be allowed on their payments for depreciation as the owners of the property are entitled to claim that they should be allowed a fair interest on their investment. All that the company is concerned about is that it should have funds in hand, obtained from rates, which will enable it to pay for renewals as they are necessary. All that the rate-payers are concerned with is that they shall have contributed each year the smallest sum necessary to bring this about, and this is the sinking-fund depreciation.

Now, when the rate-payers have paid rates high enough to allow a sinking-fund depreciation, they have no further interest, as rate-payers merely, in the question, nor is it of any consequence to them, as rate-payers, what the company does with the depreciation allowances which they have paid. It makes no difference to them whether the company puts these depreciation allowances in a fund, whether it uses them for extensions to the property, or whether it distributes them as dividends. Having contributed these sums, they are entitled to claim, and should claim, and the Courts will sustain them in claiming, that when the property or item under consideration is worn out, it shall be replaced by the company at its own expense, that is to say, without further contribution from them, as it was before, without any increase of capital on which future earnings are to be made. The rate-payer, of course, is concerned that the depreciation funds should be safely invested, so that they will be available when the time for renewal arrives, and, of course, it is for his good that the rate of interest allowed in figuring the depreciation allowance by the sinking-fund method should be as high as is safely practicable; but as long as the company must replace every item of its property in kind, as it wears out, at its own expense, and is in a position to do so without injury to the service rendered, the rate-payer has no further interest with what is done with the depreciation allowance. As a citizen, or as an investor, he may be concerned in this matter, and would object to having the depreciation allowances paid out in dividends, as this course would enable the stockholders to receive dividends greater than a fair return and then, perhaps, sell the property on the basis of such exaggerated returns to other investors who might not realize that the former owners had made no provision for renewals. Proper accounting and financing, therefore, require that only a fair return shall be paid in dividends, and that the depreciation allowances shall be invested or held in some manner which will insure their being used for the purpose for which they were paid in. This is for the interest of stockholders rather than for the interests of the rate-payers, as such.

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It will probably not be questioned, therefore, that the stockholders of a property built to-day are entitled to a fair return, permanently, on the money originally invested, if honestly and properly spent, and that the rates should also provide for the accumulation of yearly depreciation allowances. If this is so, is it not then clear that the cost-of-reproduction-new should, so far as a physical valuation is concerned, be used as the basic value on which earnings should be secured? In order to answer this, it is at this point necessary to see clearly the relation between the reproduction method and the cost-new method.

In a concern organized to-day under public supervision, with accounting methods also under supervision, there would never be any necessity for using the reproduction method or for making any valuation. Its stockholders would be allowed to earn a fair return on the cost, plus depreciation allowances; renewals would be made in kind without increase of capital, and all betterments would be charged to capital. If there should be a lack of a fair return in early years, any such deficiency would be included under cost of developing the business, and in subsequent years a return would be allowed on this also. Such a procedure would not prevent an unwise venture from failing, for an increase of rates might diminish net earnings, there might be a continued cessation of dividends, and the concern might eventually collapse; but a concern wisely planned and economically built would prosper, and its stockholders would have no cause to complain. There would be no questions as to the appreciation of land; if, through the growth of the community, the value of its lands should largely increase, the concern could not be bought out by another concern with the expectation of fair return on the increased value, because the original cost would govern.

In the case of most corporations, however, especially railroad corporations, the original-cost method is not possible, because the data are not available. What is sought now in making a valuation, is a basic physical value from which to start new and on which, in future, to base earnings. For this purpose no other method is available than the reproduction method. The cost is estimated of reproducing the property new at the present time. This is taken as the basic physical value of the property, just as the original cost, with subsequent betterments, would be taken as the basic value if that method were available. The speaker does not understand that it is at all contemplated that the reproduction method shall be applied from time to time to the same property and the rates modified according to each valuation; he does not believe in such a procedure. As he conceives it, a valuation is to be made once for all in order to get a basis from which to start new, after which, proper accounting methods will be required, renewals in kind must be made from earnings, better-

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ments capitalized, and rates can be regulated in the future without another valuation. The reproduction method is used because it is the only possible method of general application.

The kernel of the whole matter, therefore, it seems to the speaker, is the question whether the cost-of-reproduction-new should be taken and used as the equivalent of original cost. Probably few will question the statement that, when available, the original cost, allowing for deferred dividends, is a proper basis of physical valuation on which earnings should be secured. Stockholders who invest their money honestly and intelligently in a public utility concern, are entitled to receive a fair return on that cost permanently, together with operating expenses, taxes, and a depreciation allowance, and any deficiency of dividends is entitled to be capitalized. Depreciation should not be deducted in finding a physical value on which to base proper earnings. The original cost is the proper basis, and is the basic physical value on which stockholders should receive a fair return.

Now, if the cost-of-reproduction-new is also a proper basis, it must give the same result in cases where both are applicable. In the case of a new concern, the original cost would be, of course, the cost-of-reproduction-new. The same would be true in a concern in which the cost-new could be ascertained, and the books had been properly kept, and in which costs had not altered since construction. There are many corporations, however, in which the original cost method is impossible of application by reason of lack of data. In this dilemma, as the speaker understands it, we turn to the cost-of-reproduction as the only other available method. In using this method we are necessarily thrown largely on surmise and judgment, but no other method is possible. The costs have changed since the property was built, and no one can say with certainty that the cost-of-reproduction-new will be greater or less than the original cost. In the case of our railroad companies, lands were granted them by the public, and these, together with lands which they bought, have largely increased in value, mostly by reason of the existence of the railroad itself. Lands owned by private owners have likewise increased in value. It seems to be no injustice to allow the corporation the benefit of an increase in value which has come from its own existence, and which is no greater than the increase in value which its existence has brought to the property of private owners. Moreover, in the case of many of our railroad companies, if not of all of them, dividends were deferred for many years, reorganizations have taken place, and a great deal of money has been spent in betterments and charged to operating expenses; so that, as stated, the cost-of-reproduction-new cannot be said to be greater or less than the original cost, if the latter could be accurately ascertained.

The statement then is this: The reproduction-new method gives the same result as the original cost method where both are applicable; in

this case the original cost is a proper basis and would be used. Must we not, therefore, in cases where the original cost method is not applicable, assume the cost-of-reproduction-new to give the same result and to be used in the same way as the original cost would be if we could find it? The answer to this question appears clearly to be in the affirmative. Mr. Swain.

The Supreme Court, in the Knoxville decision, held that if, in the past, rates had not been high enough to provide a proper return and a proper depreciation allowance, "the true value of the property * * * cannot be enhanced by a consideration of the errors in management which have been committed in the past." This rule should work both ways; and if, in the past, rates have been so high as to have permitted excessive profits, or if the public has given lands for nothing, then the value of the property cannot be reduced by a consideration of the errors committed by the public in "the past", even if we choose to designate as errors a policy which, on the whole, has resulted in a wonderful national development and prosperity. We are starting new with a value of the property, and in the speaker's opinion, there is no method which will be uniform in application, and will reconcile all differences of conditions, except to take the cost-of-reproduction-new, and to consider that this is to be used exactly as if it were the original cost.

The first principle on which the speaker has based his conclusions, therefore, is that the cost-of-reproduction-new is to be taken and used in the same manner as the original cost. If this be true, it is perfectly clear that the undepreciated value should be the value on which a fair return should be based, just as the original cost, when available, should be the fair value on which a reasonable return should be based.

Of course, it must not be forgotten that a physical valuation is only one of the elements to be considered in deciding on rates or any other question. Neither the original-cost alone, nor the cost-of-reproduction-new alone, is necessarily the present value; but we are not discussing this question here. We are simply endeavoring to decide what physical value is the proper one to consider as the basic one on which, so far as physical value is concerned, fair earnings are to be made.

The Committee has apparently been governed in its attitude by the assumption that the Courts had finally and definitely decided that the cost-of-reproduction-less-depreciation must be used, and its endeavor has been directed toward finding a method by which the same total annual return would be received by the corporation as by using the original cost.

This result is accomplished by the Committee's method only in case the rate of return on the investment and the rate of return on the sinking funds are the same. If the sinking funds are invested in extensions of the plant, this may be true, but in many cases the sinking

Mr. Swain.

funds cannot be invested in the plant, and the rate of return on the sinking fund would be less than that received from the investment. Indeed, inasmuch as the sinking fund must be assumed to be invested safely above all things, there seems to be no reason for assuming a greater rate of interest on it than the company receives for its cash, or the rate of interest for time loans, which, in general, would be less than a fair return on the investment. In this case, the Committee's method, as shown by Column 7 of the table on page 34 of the report gives a smaller return, except in the first year, than is fair. It results, therefore, in confiscation.

The object of the Committee, however, is perfectly clear and quite praiseworthy. It assumes that two things have been finally and irrevocably settled by the Courts:

First.—That the cost of reproduction-new-less-depreciation must be used as a basis;

Second.—That the mistakes of the company in the past, in not charging rates sufficient to provide a fair return plus a depreciation allowance, cannot now be rectified.

These two findings are justified in the decision of the Supreme Court in the Knoxville case. It must be remembered, however, that the Courts, being human institutions, are subject to error; that every Court has had its decision reversed probably more than once; and that the Supreme Court of the United States has reversed its own position in a number of instances.

Much is said, in discussing this subject, as to respect for the Courts. The speaker believes that there is no one who has greater respect for them than he, but it is very important to understand clearly what respect for the Courts implies. It certainly does not imply agreeing in all cases with their decisions, or even accepting them without argument, if we believe them wrong. Judges, like those who are judged in this world, are human, and it would be less respectful to attribute to them superhuman attributes by assuming them to be always correct and infallible, than to admit their fallibility. Proper respect for the Courts means that we should believe our judges to be honest, faithful, disinterested men, honestly striving to administer justice, and that we should accept their verdicts when rendered, if we have to and are obliged to act on them; but that does not prevent us from fighting with all our might and main to convince them that they are wrong, if we believe such to be the case. The present subject is in the making, and it is quite possible, if not probable, that the Supreme Court may reverse its finding with reference to the facts referred to. Assuming those findings to be correct, however, the Committee pursues a perfectly consistent course. This may be illustrated graphically:

Mr. Swain.

In Fig. 8 let OY be the first cost of a piece of property, and OX its life, say, 10 years. The curve of the sinking-fund depreciation is shown by the curve, YX. ab is the annual payment which in 10 years will amount with interest to the original cost, or the assumed cost of reproduction. At the end of 5 years, five annual payments have been made, which, with interest, amount to $a_5\ b_5$, leaving $b_5\ x_5$ as the assumed depreciated value; $c_6\ b_6$ is equal to ab plus the interest for a year on $a_5\ b_5$; $c_6\ b_6$ is also the annual payment which in 5 years with interest will amount to the depreciated value, $b_5\ x_5$. Now, the Committee argues, assuming that the depreciated value must be taken, that the company should be allowed to earn a fair return on $b_5\ x_5$ plus an annual depreciation charge of $c_6\ b_6$, which is the same thing as a fair return on $b_5\ x_5$ plus sinking-fund interest on $a_5\ b_5$ plus the depreciation charge, ab. If, therefore, the rate of return on the property is

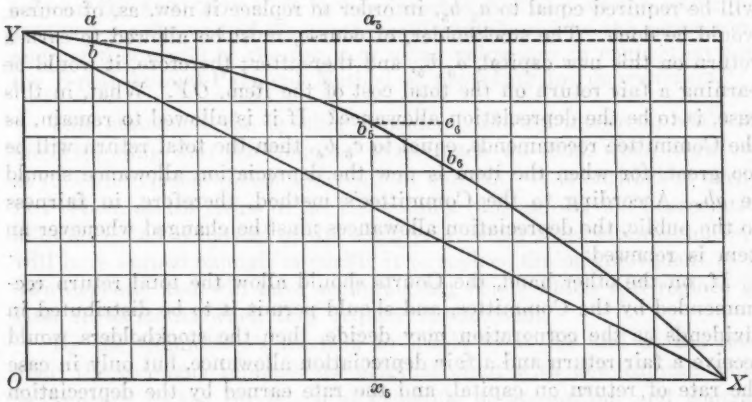

FIG. 8.

equal to the sinking-fund interest, this is the same thing as a fair return on OY plus a depreciation charge, ab. If, however, as must be assumed the fact, the sinking-fund interest is less than a fair return on the investment, then the Committee's method does not allow a fair return on the original property plus a proper depreciation charge.

If, therefore, a concern built to-day, and costing OY, is appraised 5 years from now, and if, on account of the original records being lost, or for any other reason, the cost-of-reproduction method is used, it will result in not allowing a fair return to the investor on his investment. It must be reiterated that though the rate-payers have contributed the sums which go into the depreciation fund, the stockholder has not received them; the company has received them, but it is obliged to keep them available for a specified purpose, namely, for reproducing the property when it is worn out. The statement of the

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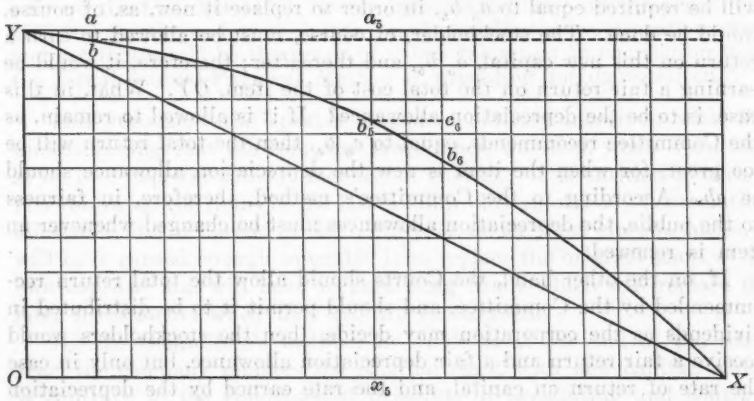

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Committee that inasmuch as the sum, $a_5\ b_5$, has been returned by the rate-payers, the company is not entitled to receive a further return on this property, is an evident fallacy, for though $a_5\ b_5$ may represent what has been paid by the rate-payers, with interest, it has not gone to the stockholder, and interest on this sum is not entered in the capital account, but in the reserve account.

Moreover, even supposing the Committee is justified in believing, or hoping, that the Courts would allow a fair return on $b_5\ x_5$ plus a depreciation allowance, $c_6\ b_6$, is it not equally probable that the Courts will require the latter to be held in reserve and not distributed as dividends? In this case the property of the stockholder is confiscated.

Moreover, if a depreciation allowance of $c_6\ b_6$ is permitted, this will allow the replacement of the property when worn out, at the point, X, only its depreciated condition, represented by $b_5\ x_5$ and new capital will be required equal to $a_5\ b_5$, in order to replace it new, as, of course, would be done. The stockholder, of course, must be allowed to earn a return on this new capital, $a_5\ b_5$, and thereafter; therefore, it would be earning a fair return on the total cost of the item, OY. What, in this case, is to be the depreciation allowance? If it is allowed to remain, as the Committee recommends, equal to $c_6\ b_6$, then the total return will be too great, for when the item is new the depreciation allowance should be ab. According to the Committee's method, therefore, in fairness to the public, the depreciation allowances must be changed whenever an item is renewed.

If, on the other hand, the Courts should allow the total return recommended by the Committee, and should permit it to be distributed in dividends as the corporation may decide, then the stockholders would receive a fair return and a fair depreciation allowance, but only in case the rate of return on capital, and the rate earned by the depreciation fund, were identical; if the latter were less than the former, as must be assumed, the property of the stockholder would be confiscated, for it would not receive a fair return.

All this uncertainty and complication will be avoided by allowing a fair return on the cost-of-reproduction-new, and a fair depreciation allowance on this same sum (namely, ab), which will renew the item, if new, in the term of its life, independent of its age when the valuation is made.

It seems reasonable to assume that the Courts will finally see the truth of this, if we insist upon it, and make it as clear as we can.

It is true that the method recommended by the Committee is fairer to the corporation than that which has often been followed. It is much fairer for the corporation to be allowed (referring to Fig. 8) a fair return on $b_5\ x_5$ plus a depreciation allowance of $c_6\ b_6$ than to be allowed only a fair return on $b_5\ x_5$ plus a depreciation allowance of ab. But the speaker maintains that the investor is entitled to

a fair return on OY plus a depreciation allowance, ab. He believes that the Committee loses sight of the fact that the question is not that of following a piece of property throughout its life, but of jumping in at some intermediate period, as at x_5, with no knowledge of original cost, and determining at that time the value on which fair returns should be allowed. Mr. Swain.

The Committee, in its illustrations in the Addendum, assumes that in each year precisely the depreciation allowances are invested—no more and no less—in extension of the plant. Refer, for instance, to Tables 5 and 6. Table 5 represents the sinking-fund depreciation invested in the plant, and the value-new is given in the last column. The Committee says:

"It would be grossly unjust to the rate-payers to permit the owner of the property, under the conditions presented, to earn dividends, not only on the $100 000 which he has invested, but also on the additional $177 609 [at the end of the ninth year] contributed by the rate-payers, and yet this would be the result of using value new in connection with the Equal-Annual-Payment Method of providing for depreciation."

This statement the speaker considers entirely incorrect, with the understanding that the company must renew an item, when worn out, at its own expense without increase of capital. It is entitled to earn a fair return on the items in the last column plus an original depreciation allowance, and at the end of the tenth year, under these circumstances, it will have earned enough to enable it to replace the original plant at its own expense, and will have accumulating funds which will replace the later investments when they require replacement.

By the Committee's method, in Table 6, at the end of the tenth year, the company replaces the original plant with new capital, which is contrary to the fundamental principle of accounting and contrary to the rules laid down by the Interstate Commerce Commission, and indeed contrary to all sound principles of financial management. Of course, it is easy to see what the Committee has done. It has assumed that the original plant has been exchanged for the new plant which has been built from the depreciation allowances, and that instead of selling these additions and putting the money into replacement of the original plant, it simply invests its new capital in renewal. This illustrates, however, in the speaker's opinion, the confusion which exists in the report; the illustrations are ideal, not real; theoretical, not practical. No concern would be able to invest each year, beginning with the second year, precisely the depreciation earnings in extensions. Indeed, it is illogical and misleading to confuse the question by assuming that depreciation earnings are invested in the plant. So far as the rate-payer is concerned, it makes no difference what is done with the depreciation earnings. If we can decide correctly how to treat the

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original plant, we shall then know how to treat any additions to that plant. The original plant should be kept separate in our consideration, and the fundamental principle must be maintained that it must be renewed by the company, when worn out, at its own expense. The Committee's method of treating the subject is misleading and confusing in this respect.

The speaker has taken Table 5 and extended it, showing what the result would be, if, as he maintains, the company should be allowed to earn a fair return on the value new plus a depreciation allowance. The result shows that at the end of 10 years there is an amount available which will be sufficient to replace the original plant without new capital; it also shows additional sums equal to the proper depreciation allowances necessary to replace the extensions which have been paid for from depreciation earnings.

Much has been said about the depreciation fund, and the question now arises, how large a fund is necessary in the case of an ordinary public service corporation. Such a corporation, after a few years of operation, settles into what may be called a state of financial stable equilibrium. Just about the same sum will be needed each year for renewals, and the only depreciation fund, or reserve, that is necessary is one which is large enough to take care of the yearly inequalities in these renewals. A railroad company, for instance, with hundreds of thousands of ties, thousands of tons of rails, thousands of bridges, stations, cars, locomotives, and other items of property, requires practically a nearly uniform sum each year for maintenance and renewals. In one year it may have to renew more bridges than usual; the next year it may have to renew less, and so with the other items. A comparatively small sum is all that is needed as a depreciation fund or reserve, and the larger the property—that is to say, the more numerous the individual items—the less the fund will have to be. This fund is simply in the nature of a reservoir to balance annual inequalities, exactly as the reservoir for the water supply of a city is required to balance the unequal draft at different hours of the day or in different months of the year. The more uniform the draft and supply, the smaller the reservoir may be. In other words, the depreciation allowances set aside for any given item of property in any given year are at once paid out to renew other items which require renewal in that year.

The speaker wishes also to call attention to another source of confusion in the report of the Committee, and one which is also to be found in many reports and decisions. It arises from confusing two meanings of the word "depreciation". Properly understood, depreciation means a loss of value due to wear and use, or to approaching obsolescence. This is the kind of depreciation that we have

Mr. Swain.

been considering. The word is often used, however, to mean also the lessening of value due to the fact that a given piece of property might be produced to-day at a less cost than it could be produced for when new. This is an entirely different thing, and the same term should not be used for it.

When it is claimed that the cost of reproduction new should be used as a basis for rates, the reply is often made by those who have not considered this distinction: "That cannot be correct; you allow the appreciation of land because it has increased in value; why should you not allow the depreciation of those elements which have decreased in value; one is the opposite of the other; you should allow both or neither."

This argument is fallacious, as will clearly be seen by considering the foregoing distinction. The reason we allow the appreciation of land is because we are using the cost-of-reproduction method, and, in using that method, we do allow for "depreciation of things which have depreciated". That is to say, every element of property which costs less to produce to-day than it did originally we put in the inventory at its cost to reproduce to-day, just as we put in the land at its cost to reproduce to-day. Items of property which cost more to reproduce to-day than they did originally we put in at their present cost; items which cost less to reproduce to-day than they did originally we put in also at their present cost; we make no difference. The question of depreciation due to wear has nothing to do with this; it is an entirely different one. Instances of this fallacy will be found in the Committee's report, and also, for instance, in the decision quoted by the Committee on page 51, which, instead of being clear, as the Committee states, the speaker considers to be quite confused in its reasoning. For instance, on page 50, the Committee says:

"The Courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value. The principle involved in the two cases is the same, and there is no reason to think that its application to both would not be supported by the higher Courts."

The speaker respectfully contends that the principle involved in these two cases is not the same, but is entirely different.

The question of depreciation due to wear, then, is not to be set in contrast with the appreciation of land, and it is this depreciation which we are considering. If we consider the cost-of-reproduction method, we must put in every element, land included, at the fair cost of reproducing it to-day, whether it is greater or less than its original cost.

Mr. Swain.

In view, therefore, of the arguments adduced, there is no question in the speaker's mind that in valuation for rate purposes the undepreciated value should be taken, unless we are ready to admit that the cost-of-reproduction-new is not to be taken as a figure which replaces the original cost, which cannot be ascertained. If, however, we assume that the reproduction value is not to be taken in place of the original cost, then we find ourselves in the logical inconsistency, when we use the reproduction method, of using a method which does not give the same results as an admittedly correct one (namely, the original cost) in cases where both are applicable.

TABLE 24.

(1) Year of operation.	(2) Percentage on investment earned for distribution to investors and for replacements and renewals.	(3) Percentage paid to investors.	(4) Percentage remaining for replacements and renewals.	(5) Depreciation in percentage.
1st	5.00	5.00	0.00	4
2d	5.75	5.75	0.00	4
3d	6.50	6.50	0.00	4
4th	7.25	7.25	0.00	4
5th	8.00	8.00	0.00	4
6th	8.80	8.00	0.80	4
7th	9.60	8.00	1.60	4
8th	10.40	8.00	2.40	4
9th	11.20	8.00	3.20	4
10th	12.00	8.00	4.00	4
Total for 10-year period	84.50	72.50	12.00	40
Average for 10-year period	8.45	7.25	1.20	4

Table 24 will make clear the fact that to take the cost-of-reproduction method, using the value-new-less-depreciation, may, in some cases, mean confiscation. Suppose, for instance, that a public utility property is constructed and put into operation—let us say, an interurban street railway. Suppose that all the money is expended properly and that the road competes with a steam road and charges rates, say, 10% less than the latter, in order to attract business. Suppose there is no need of expenditure for renewals for the first 10 years, after which renewals begin, and soon attain a state of stable equilibrium. Suppose that the percentage earned on the investment which is available for distribution to investors and for replacements and renewals—that is to say, the amount earned above operating expenses and taxes—is as shown in Column 2, varying from 5% in the first year to 12% in the tenth and subsequent years. Suppose that investors receive the percentage in Column 3, varying from 5% in the first year

up to 8% in the fifth year and in each year thereafter. Then the percentage remaining for replacements and renewals will be as shown in Column 4, being zero for the first 5 years, and increasing to 4% in the tenth year and thereafter, the total accumulation in 10 years being 12% on the capital. Suppose that the depreciation is calculated by the straight-line method, taking a life of 25 years, or 4% per annum, making 40% in the 10 years. Mr. Swain.

This property, then, at the end of the 10 years, is earning 12% and gives every indication of continuing to earn that amount, of which 8% is paid to the investors and 4% remains for replacements and renewals, in addition to the fund of 12% which can be drawn on for the same purpose. The property is successful, and although the original cost was $1 000 000, its stock sells for about 120; that is to say, is bought and sold between investors on the basis of $1 200 000.

Now, suppose that, for some reason or other, the original records have been destroyed, perhaps by fire, and that an appraisal is to be made at the end of the tenth year. The cost-of-reproduction method is used and shows a cost-of-reproduction-new equal to $1 000 000; but the commission says we must take the depreciated value, and the depreciation is about 40%, so that as a basis for earnings, the value of the property is only $600 000. To this we may add the 12% in the renewal fund, or $120 000, making a total of $720 000. A similar result is reached if we use sinking-fund depreciation. The original stock paid 8% and called for a distribution of $80 000 annually. If 8% is now only to be allowed on $720 000, it will call for $57 600 annually, or 5.76% on the par value.

Under these circumstances, the value of the stock in the market will drop from 120 to perhaps 80 or 90, and if there is danger that some public authority will take the property on the basis of a value of $720 000, the stock may drop to 72, or even lower. In other words, the value made on the basis of cost-of-reproduction-new-less-depreciation, would confiscate the property of the stockholders, which was selling in the markets at $1 200 000 and justified that value, and now becomes worth only about $800 000, or perhaps less. If this is not confiscation, then the speaker does not know what confiscation is.

This issue must be faced. The Committee, in the table on page 34 of the report, shows a progressively decreasing return on the value of the property, and, on page 49, it says:

"It is true that of the return on a given item of property, the part applicable to dividends becomes less as the property grows older, but this is as it should be; a part of the capital invested in that item has been returned and has either been invested in additions or replacements, or temporarily in a depreciation fund, so that the whole of the original investment is at all times earning returns applicable to dividends".

Mr. Swain.

As has been already demonstrated, it is only when the rate of return on the property, and the rate of return on the depreciation fund, however used, are the same, and if the depreciation allowances may be divided, in part, among the stockholders as dividends, that justice will be done; otherwise, there will be confiscation.

This report will be understood to mean, the speaker believes, that dividends should decrease progressively, or that, if a valuation is made of an admittedly successful and honestly financed concern, like the one used in the previous illustration, then dividends must be reduced on the basis of that valuation. Does the American Society of Civil Engineers, or a Special Committee of the American Society of Civil Engineers, wish to advocate such a procedure? And if so, for what reason? Merely because the Courts may have made mistakes in the past; and to conform to an ideal and unreal point of view taken by the Committee. This report will postpone the day when public service commissions, Courts, and the public will perceive that a continuing concern, honestly serving the public at fair rates, is entitled to dividends equal to a fair return on the full amount of the investment; and by assuming, as a part of the argument, that all depreciation reserves are invested in extensions, it will obscure the fundamental principle that property which wears out should be replaced by the company in kind at its own expense.

If thus interpreted, it may do great injury to public utility companies, and so to the public at large.

There is one quotation which the speaker wishes to get into this discussion somehow, if Mr. Humphreys does not include it in his remarks. It is from a brief by Charles F. Mathewson, of the New York Bar, in the case of the Kings County Lighting Company *vs.* The Public Service Commission, and Mr. Humphreys says it is the most logical statement he has read. Mr. Mathewson says:

"The proposition [to deduct 'accrued depreciation' in valuing plants in rate-making] is so absurd on its face that it hardly needs discussion to show its fallacy. Why, aside from the question of 'confiscation', should consumers, for exactly the same service, equally efficiently rendered, expect to pay less in the sixth year than in the first year, merely because some items of plant will (viewed at the sixth year) require replacement at a date in the future then nearer than such date was at the beginning of operation? As well might it be claimed, to repeat a homely illustration, that a farmer should regulate the price of the eggs which he sells, by the age of the hen which lays them—reducing the price of the product as the hen gets on in years. The reason he does not is that the service efficiency and operating value of the hen, as evidenced by the quality of the eggs which she lays, are not impaired by the fact that her life is advancing. That advancement may concern the farmer and possibly concerns the hen; but it in no manner affects the value of the eggs to the consumer, or justifies

Mr. Swain.

him in demanding them at a lower price than he paid at an earlier period of her life. The consumer of the eggs must expect to pay a sufficient price to afford a return to the farmer on his total investment in the hen during her life, *plus* enough more to enable the farmer on her death to replace her, and thus keep his investment unimpaired. A farmer could hardly be expected to invest in hens for the purpose of supplying the public with eggs, if for a portion of their life he was to receive a return on only a third or a half of his investment; and any such rule would simply compel the public to go without eggs until the regulating power (if such there were) saw fit to revise its reasoning. There is absolutely no difference in the economic principles applicable to the operation of a gas plant and the operation of a hennery, so far as concerns right to return on capital; and what is absurd in one case is equally absurd in the other. The fact that the rate of return in the one case is subject to reasonable regulation, and not in the other case, has no bearing on the main proposition."

Mr. Mathewson is here arguing against the proposal to allow the stockholder only a fair return on cost of reproduction-new-less-depreciation.

The speaker wishes to say one word in regard to Mr. Snow's statement, in order not to be misunderstood. The speaker did not say that the reserve funds should not be invested in the property, but did maintain, and still maintains, that it confuses the main question to assume that they are necessarily going to be invested in the property. They need not be, and in some properties there is no expansion for many years. It confuses the question to assume that they are going to be thus invested. They must be invested so as to bring a return, and the only obligation the company is under is to replace every element, as it wears out, in kind, at its own expense. The speaker wishes to be distinctly understood that he did not say that it was an improper use to invest the reserve funds in the property; and he also called attention to the fact that, in most public utility corporations, such as railroads and street surface railways, a small fund will provide for all necessary contingencies; that is, for the variation in the renewals from year to year, only a small fund is required, and that is all that need be held.

Mr. Ledoux.

J. W. Ledoux, M. Am. Soc. C. E. (by letter).—In regard to the valuation of utilities for the purpose of rate-making, it would be desirable if engineers could agree on general rules which could be followed in all cases with the least injustice both to the owners of these utilities and the public.

In these questions most people look to Court decisions for their authority, but the Courts have not settled the matter definitely, and they, as well as those versed in the law, cannot be expected to have

Mr. Ledoux.

the special training to handle this particular problem as well as technically trained and experienced engineers free from bias.

The writer is of the opinion that the members of the Engineering Profession are freer from injustice than those of most other professions. Unfortunately, however, partisanship does exist, and it is common to find municipalities or private corporations looking for engineers who know how to, and will, make a strong plea for their side of the case. This phase, in an exaggerated form, was illustrated by a prominent engineer who was heard to say recently that "If a consulting engineer, in his public testimony, is known to have given a corporation fair treatment, his 'goose is cooked' forever afterward as far as municipal employment is concerned". This condition tends toward shiftiness and ambiguity in the discussions of some of those whose standard of justice is subservient to their own pecuniary interest.

There are comparatively few points on which engineers should differ, and it is believed that they would be in perfect accord in all the principles if they were agreed as to the premises.

This discussion is written from the standpoint of a water-works plant, and is based on what the writer considers, at the present moment and after careful reflection, to be abstract principles of justice. These conclusions were reached after the successive abandonment of several materially different opinions which, at the time, were held with more or less assurance. He realizes that as now drawn they are not strictly in accord with some Court decisions which necessarily must be the gospel, but, in their final analysis, they would be based on the concensus of opinion as to what constitutes fair dealing.

If a public utility were being taken over by a municipality by the process of condemnation, the rates and revenue being settled, it is evident that the value of the utility should be paid. If, however, a public commission has in hand the establishment of rates for a utility, the question of condemnation being non-existent, then the cost of the utility should be determined, in order to find the proper and just fixed charges. In the case of condemnation, if the municipality has the right to establish its own works, and the present plant is adequate, the cost of reproduction less depreciation plus the value of the established revenue should be the criterion. In the case of establishing rates, the cost of the existing plant should be the criterion—not necessarily what it cost the owners, but what is reasonable and proper under the circumstances in which the existing plant was built and developed. Where there is doubt, the recorded book cost should be the guide.

As to the items making up this cost, there should not be material difference between experienced and intelligent opinion. Those things which cannot generally be avoided in average or ordinary practice should be allowed—the cost of preliminary engineering; legal, finan-

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cial, and other expert reports; cost of promotion, organization, surveys, designs, land, rights of way, water rights, title examinations, and management, are some of the items which come in before any work is done. Then there is the cost of construction; liability and other insurance; interest during construction; architectural, chemical, legal, engineering, and other expert fees and commissions; cost of lawsuits; most of which items and more are practically certain to occur. There should be added, also, an item for contingencies to cover things which cannot be foreseen, but which, in some form or other, are sure to occur, such, for instance, as weather difficulties, errors, accidents to plant or materials of construction, delays and extra expense due to various causes, such as non-arrival of material, injunctions, obtaining of permits, changes of grade, caving, settling of embankments and foundations, defective dams or reservoirs, failure of sub-contractor, strikes, and many other things which cannot be specifically foreseen, but which should be allowed for in a general percentage based on experience. There are also the items of financing, underwriting, and general contractor's profit.

Then when the works are finished and capable of supplying service to the community, it generally takes 10 years or more before enough contracts can be obtained to pay current expenses. During this period of waiting, which is common to the majority of plants, when, also, there may be further improvements, construction, and extensions, which, of course, must be allowed for, the owners would have to pay out much more than they receive, and this deficiency of receipts is a necessary item to figure in the development cost of the plant.

It is often held that the valuation of a plant should be the same whether the question of rates or purchase be considered. Theoretically, the position is unsound, for in the case of rates we have a property which properly represents a given investment, on the basis of which rates should be established to yield a revenue profitable to the owners. In the case of purchase there is a given revenue with a given property to determine its value. This revenue may make the property a losing or a profitable venture, and, manifestly, the value of the property for purchase is directly influenced by this fact.

The proposition that the cost instead of the value should be taken in establishing rates may not be accepted immediately, on the ground that, value being established regardless of cost, if any other basis be taken, the property is subjected either to inflation or confiscation. However, from the moment it is decreed that the State has the right to fix the rates of a utility, its power to fix value follows as an inevitable consequence.

It will be generally conceded that the State should recognize as just such revenues as would pay the owners of a utility a reasonable

Mr. Ledoux.

return on their investment, but here we are confronted with another difficulty. The investment of the present owners of the utility may have been based on value instead of cost. They may have paid for the utility, by reason of its existing or prospective net revenue, more or less than the cost of the actual items entering into the creation of the property. If, however, these existing net revenues were in all cases just to the utility and the public, there would be no reason for State supervision of rates.

It may be assumed, therefore, that the State would look on such rates as unjust, that would yield an undue return on the reasonable cost of the existing utility. As illustrative, a municipality invites a concern to establish a water-works, the character of which and the water supply being definitely specified. The necessary investment and the rates are agreed on. After some years, the municipality decides to build its own works, possibly at less cost, and, being freed from such expenses as taxes, licenses, permits, etc., the cost of operation will thereby be reduced. Prior to this decision, the net revenues paid a reasonable return on the investment, but, fearing competition and a deficiency of revenue, the original company sold its property to a new company for half the cost. Then a rate-making commission is established, and is called on to determine rates. As an opposite case, a water-works is established in the same manner. The growth and density of population and the net revenues became so great, coupled with the fact that they own the only available water supply, that the original owners are enabled to sell to a new company for double the amount of their investment; after which the rate-making commission is called in to establish new rates. In both these and all such cases it would seem that the criterion should be the proper and necessary investment to produce the actual property, regardless of past trading and stock jobbery.

As a general proposition, where there is no contract, it would be desirable and equitable if the value of all public utilities were identical with the proper and necessary investment, so, if the rate-making commission is presented with a case in which the rates are so high or so low as to produce revenues which make the value different from the cost, should it not readjust these rates to the proper basis? The existence of iniquity and the investment therein does not establish the right to its continuance.

In the abstract, value varies as revenue, which varies as rates, which should vary as cost, and, consequently, value should vary as cost; but if the commission does not have the right to establish rates, then value varies as revenue, revenue varies as rates; but rates depend on "what the traffic will bear", and not on cost; therefore, in this case, the cost and value have no direct relation.

Mr. Ledoux.

Heretofore, in cases of condemnation by the municipality, or arbitration of the value of the utility, it has been held with justification that an important element of valuation was "going value", which represented what a purchaser could afford to pay for a given plant with revenue, above what the same plant was worth where the purchaser was obliged to wait for the acquisition of that revenue. In such cases this item is truly an element of value, but, in rate cases, it cannot be considered in the same way, because the rates have not yet been established on which to figure the present worth of the deficiencies of revenue according to an assumed rate of revenue increase. For rate purposes, it would be more proper to call this item "development cost", which may be defined as the necessary net outlay over and above the cost of the physical plant up to the time it is earning a reasonable return on the total investment. The practical outcome of this, however, is the same as that of "going value".

In rate cases, if one must determine this item by calculation, it will be necessary to cut and try. First, assume this item, a revenue, and a rate of increase. From the fixed charges, operating expenses, and assumed revenues, the present worth of the deficiencies of revenue can be calculated. If that does not agree with the assumed development cost, try another value, until finally there is an agreement. If the assumed revenue is more or less than enough to pay a reasonable return on the calculated total cost, a new value for revenue must be assumed, using the same rate of increase as before. Proceed in this manner until the calculated results agree with the assumption; after which, one may be as close to what it ought to be as though he had guessed at it in the beginning.

In many lines of investigation, the snap judgment of a person having broad experience is more reliable than the most abstruse calculation.

Some argue (and generally with justification) that this element of cost should be based on the actual experience of the plant under consideration, but this plant might be grossly mismanaged, more than ordinarily well managed, or the personnel of the operating force may antagonize the patrons; at any rate, it does not seem wholly proper to base cost on the reluctance or alacrity of the consumers to patronize the utility.

If the foregoing reasoning is sound, then the cost to reproduce the given plant will be taken as the basis on which to determine rates, and not the present rates as the basis of determining value, and, as a consequence, new rates.

Now, if the cost, or what is the same thing, the necessary investment, is the proper basis for the commission to adopt in establishing rates, how is this to be determined?

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There is justification for a method which will wipe out past history entirely, take the plant just as it exists, and calculate what such a plant would reasonably cost to construct and develop up to the present. This method would depend entirely on the appraiser's judgment, and this is where such a committee as that of the American Society of Civil Engineers could formulate certain rules of practice which would be of great aid to the appraiser, and which might become a recognized standard by the commissions and the Courts. These rules of practice should be determined, it appears to the writer, by studying carefully the history of twenty or more representative plants in towns having populations of from 2 000 to 1 000 000, and using the data thus obtained for calculating the uncertain and difficult elements of the plant under consideration. The cost of reproduction of the physical plant is neither difficult nor uncertain, but the estimate of the cost of development is the element that should be derived by the method suggested, and, in fact, the Committee might go farther and formulate some scheme such as the following:

First, estimate the cost of the physical plant and property as it exists, taking average prices which have been current for the past 10 years. To the sum thus obtained should be added certain percentages for each of the following items: Promotion and organization expenses, engineering, financing, legal expense and lawsuits, contingencies, and general contractor's profit, and from this total sum should be deducted the cost of constructions which are obsolete or of no value in the plant. The remainder should be the cost to reproduce the physical plant as it stands; there should also be added a general percentage for errors and constructions which are useless, but which are practically sure to occur and impossible to avoid in the majority of plants. Then there should be added a general uniform percentage of from 15 to 40% for development, and this percentage should be determined from the analyses of the twenty or more water-works referred to.

When these percentages are once determined and recognized as reasonable, this method would seem to be the most satisfactory that can be devised for arriving at the proper cost, and in a majority of cases it will be fair both to the owner of the utility and the people. After the cost is once derived in this way, it only remains for the commission to establish such rates as will produce a revenue sufficient to pay all operating expenses, fixed charges, depreciation, and a reasonable return to the owners of the utility.

Many engineers would object to this method on account of the arbitrary fixing of certain percentages to allow for uncertain elements of cost, but, no matter what allowances are made, they must necessarily be based largely on speculation, and it is an advantage, where speculation cannot be avoided, for all engineers to speculate uniformly, for then we will not be confronted with cases like one which came

to the writer's knowledge a few weeks ago, where, in the valuation of a water plant for establishing rates, three experienced engineers estimated the "going value"; two made it some $30 000 odd, and the third more than $130 000. Mr. Ledoux.

At first sight this method would appear to give the recently established water company an undue advantage by reason of its not having incurred the expenses of development belonging to one established for more than 10 years, but these expenses are sure to occur later, and the rates should be such as would yield the desired revenue when the consumers are connected up at the end of the development period.

The subject is one of great difficulty, and no method can be devised which will not in some exceptional cases be apparently unsatisfactory, but, until some recognized method is adopted fair to the utility, investors will not be willing to embark in propositions of this kind, and the people will be the ultimate losers.

It will be observed that in this discussion no deduction has been made for accrued depreciation. From the moment a plant is built, depreciation, an insidious enemy, begins to rob it of its substance and carries away every year a definite portion of its value. If the owners of the plant were responsible for the robbery, they should not receive credit in valuation for the property thus abstracted, as, for instance, if they sold every year a definite portion of the plant and kept the proceeds for their personal use. On the other hand, if the public, or people, living along the lines, for whose benefit or use the utility was constructed, were responsible for this annual abstraction, then the owners of the plant should be reimbursed by the people. When the plant was built, however, both parties recognized the existence of this enemy and the extent to which his maraudings would inevitably reduce the value of the property each year, and the necessity of maintaining by replacement as required the property thus lost. The money to pay for the cost of these replacements can come from only two sources: the owners of the utility or the people. If it comes from the owners, unless they be the people, there is no return for the money thus expended. Unless the utility be a philanthropy, the owners must be reimbursed at least for all legitimate and unavoidable expenditures, for even then the undertaking is profitless. This reimbursement can only come from the revenues produced by the rates charged the people for the service rendered. As this abstraction of value by depreciation neither reduces the amount of the investment, nor inures to the benefit of the owners, therefore, in valuation for rates, no deduction should be made from the proper cost of the property for accrued depreciation.

It will be asked, how is one to insure that the owners of the plant will provide out of the revenues thus produced an inviolable fund

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for depreciation, instead of pocketing this allowance or paying it out in dividends, and letting the plant gradually go to destruction?

If the revenues produced from the rates established by the commission are large enough to pay operating expenses, taxes, interest, depreciation, and profit on the legitimate original cost of the works, and the owners of the plant do not invest this depreciation fund in the plant, it has lost just so much value or investment cost, which has gone to make the profits unduly large. Consequently, an equivalent deduction in cost should be made when the commission again establishes rates.

The same principle should apply the first time if it is found that the previous revenues have been high enough to pay all expenses, profit, and depreciation, while the plant has been allowed to depreciate with nothing to show for a restoration fund.

For purchase, the case is entirely different, depending on the conditions outlined in the four following cases:

Case 1.—The city condemns, the rates being fixed;

Case 2.—The city and owners of the utility mutually agree to an arbitration, the rates being fixed;

Case 3.—The commission establishes the value of the plant at which it will be taken over by the city, the rates being under the control of the commission;

Case 4.—The city purchases by mutual barter, the rates being fixed.

In each of the first three cases, according to the writer's judgment, depreciation should not be deducted unless the company has been earning sufficient to pay all expenses, including depreciation and a reasonable profit, and no restoration fund exists.

In Case 4 depreciation would naturally be deducted, at least by the representatives of the municipality.

The question of street pavements also leads to confusion of thought: If the works were built when no pavements existed, and these were afterward constructed by the city, should the rates be increased thereby, and, in case of purchase or condemnation, should these pavements be an element to increase the value of the utility? In the case of rates, the municipality has added a feature which increases the cost of maintenance and the final duplication of the piping system and, to that extent, the revenues and rates should be increased.

In case of purchase by barter, where the municipality has no right to condemn, pavements certainly add to the value of the utility on account of the greater cost to the municipality to duplicate the piping system. In case of condemnation by the municipality, or the establishment by the commission of a sale valuation, it is a serious question whether the commission should recognize as just an element of value that was created by and at the expense of the people.

ALBIN G. NICOLAYSEN, ASSOC. M. AM. SOC. C. E.—The proper treatment of depreciation is one of the most important questions in the valuation of public utilities. It is also one that has a tendency to become too involved to be followed easily if presented in general terms, and the speaker will not attempt to do this, although his conclusions were originally reached in that way. It is his intention to show how depreciation affects the value, for rate-making purposes, of the physical property owned by two imaginary companies operating under assumed simple conditions. In spite of the limitations of such a method, it will be found that the conclusions reached will apply quite generally, and, in addition, will be of material assistance when the examination is extended to companies operating under more general and more complicated conditions. Mr. Nicolaysen.

The first imaginary company which will be investigated, is assumed to operate a ferry-boat acquired at a cost of $100, and having a useful life of 10 years. If the company has no other investments, and if it is further assumed that it is entitled to earn 6% on its capital stock, and that the amount in the depreciation fund can be invested in such a way as to earn also 6% interest, then the rates should be high enough to return annually the sum of $13.587 over and above all operating and maintenance expenses, including insurance, taxes, etc., and are to be used only for depreciation and dividend payments.

If this is the case, the company is able to pay annually $6.00 to the stockholders and $7.587 to the depreciation fund. What the stockholders received was 6% on their investment, and what was paid to the depreciation fund, at the end of 10 years, under the assumed conditions, would amount to $100, enabling the stockholders to buy a new boat like the first one and continue operations under the original conditions, or to withdraw from the business with their capital unimpaired.

This shows that the combined dividend and depreciation payment in the assumed case, under fair management, should be $13.587 per year, and it is obvious that this amount is independent of the disposition made thereof. The company, if it so decided, could neglect to establish any depreciation fund, and pay out all the net earnings to the stockholders. This would not mean any increase in the return on the capital. The stockholders would receive 6% on their investment as before, and the amount paid in excess thereof, would be a repayment of capital, which in this way would be entirely returned at the end of the tenth year, when the boat would have no value.

Thus it appears that where replacements are paid for by the stockholders, the annual return on the physical property should be $(A + B)\%$ of its "value new", where A is the rate of interest on capital invested, and B is the rate of depreciation figured on the

Mr. Nicolaysen.

sinking-fund basis. This return is the same whether or not a depreciation fund is actually established, and should run undiminished through the assumed life of the property.

The correctness of this statement is confirmed by an examination of Tables 25 and 26. The value of the depreciation fund at the end of each year is shown in Table 25; where no depreciation fund is established, the investment of the stockholders is gradually reduced, as shown in Table 26, which gives the amount of the investment for each year.

TABLE 25.—SINKING-FUND DEPRECIATION. DEPRECIATION FUND CREATED.

Physical property costing $100 and having 10 years life. Annual net earnings $13.587: Stockholders receive 6%; remainder placed in depreciation fund and earning 6% per annum.

Year.	Capital invested.	Return to stockholders.	Depreciation Fund.		
			Payment at end of year.	Interest during year.	Value at end of year.
1	$100.00	$6.00	$7.587	$0.00	$7.59
2	100.00	6.00	7.587	0.45	15.63
3	100.60	6.00	7.587	0.93	24.15
4	100.00	6.00	7.587	1.45	33.19
5	100.00	6.00	7.587	1.99	42.77
6	100.00	6.00	7.587	2.56	52.92
7	100.00	6.00	7.587	3.17	63.68
8	100.00	6.00	7.587	3.82	75.09
9	100.00	6.00	7.587	4.50	87.18
10	100.00	6.00	7.587	5.23	100.00

TABLE 26.—SINKING-FUND DEPRECIATION. NO DEPRECIATION FUND CREATED.

Physical property costing $100 and having 10 years life. Annual net earnings $13.587, all paid out to stockholders; 6% considered as interest on investment, remainder as return of capital.

Year.	Investment at beginning of year.	Interest on investment.	Amount received by stockholders.	Investment reduced by:	Investment at end of year.
1	$100.00	$6.00	$13.587	$7.59	$92.41
2	92.41	5.55	13.587	8.04	84.37
3	84.37	5.07	13.587	8.52	75.85
4	75.85	4.55	13.587	9.04	66.81
5	66.81	4.01	13.587	9.58	57.23
6	57.23	3.44	13.587	10.15	47.08
7	47.08	2.83	13.587	10.76	36.32
8	36.32	2.18	13.587	11.41	24.91
9	24.91	1.50	13.587	12.09	12.82
10	12.82	0.77	13.587	12.82	0.00

Mr. Nicolaysen.

Although Tables 25 and 26 show clearly enough that the same amount is required to meet the fair claims of the stockholders, whether or not a depreciation fund is established, as a matter of policy such a fund should always be created by companies operating under conditions in any way similar to those assumed, as this is the only way in which such a company can be reasonably sure of the continuity of its enterprise and avoid the use of antiquated equipment.

The second imaginary company to be investigated is assumed to begin operations under exactly the same conditions as the company previously described, the difference between the two being that the first operates one boat only, and the second, through the opening of new routes, increase of business, etc., is enabled to add one new boat to its equipment at the end of each of the first 9 years, so that it begins its second year with two boats, of which one is new and the other 1 year old; its third year with three boats of which one is new, one is 1 year old and one is 2 years old, and at the beginning of the tenth year, it operates ten boats, one just acquired and good for 10 years' service, the next, 1 year old, and able to serve 9 years more, and so on down to the original boat which has given service for 9 years and has a remaining useful life of only 1 year.

Each of these boats, as in the case of the first company, is assumed to cost $100, to have a useful life of 10 years, and to earn, during every year of its operation, the sum of $13.587 net, available for depreciation and dividend payments.

It is further assumed that the limit of the profitable expansion is reached with the tenth year, and that, from then on, the number of boats operated remains stationary at ten, one new boat being purchased annually to replace the oldest boat in service, so that the equipment operated during the eleventh year, twelfth year, etc., will correspond exactly to that operated during the tenth year.

It has been shown in the case of the company operating one boat, that net operating earnings of $13.587 was sufficient to satisfy the fair claims of the stockholders, as it allowed the company to pay 6% dividends and create a depreciation fund which, at the end of 10 years, when the boat must be replaced, amounts to $100, the original investment.

Now it is evident that the fair return per boat is independent of the number of boats operated, so that ten times $13.587, or $135.87, must be a fair return for a company operating ten boats, and provides both for depreciation and dividend payments. In the case of the company now under consideration, the condition of the equipment in the tenth year is maintained indefinitely by the annual replacement of one boat at a cost of $100. This shows that the annual depreciation is $100 and that the portion of earnings from operation available

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for dividends is only $35.87, when the replacements are paid out of earnings. As the total "value new" of the equipment is $1 000, the interest on "value new", at 6%, is $60 in place of the $35.87 available for dividends; the latter amount is 6% of $597.80, which is thus the depreciated or present value on which interest should be figured.

The fact that earnings from operation are insufficient to provide both for the renewals of worn out boats and for a return at the assumed rate on "value new" of the physical property, does not prove that any portion of this value has been confiscated. It simply means that the total earnings include the return on the depreciation fund, created during the years when no renewals were required, and the combined earnings from operation and the depreciation fund—as shown in Table 27—will be found to be large enough to meet renewal charges and to yield the assumed return on the full "value new" of the equipment.

TABLE 27.—SINKING-FUND DEPRECIATION. DEPRECIATION FUND CREATED.

Physical property consists of boats costing $100 each and having 10 years life. One new boat purchased at the end of each year. Ten boats owned in the tenth and following years. Annual net earnings, $13.587 per boat operated. Stockholders pay for the first ten boats and receive 6% dividends. Remainder of earnings placed in depreciation fund that earns 6% per annum and pays the cost of renewals.

Year.	Capital invested.	Net operating earnings.	Return to stockholders.	Depreciation Fund.			
				Payment at end of year.	Interest during year.	Paid out for renewals.	Value at end of year.
1	$100.00	$13.587	$6.00	$7.587	$0.00	$0.00	$7.59
2	200.00	27.17	12.00	15.17	0.45	0.00	23.21
3	300.00	40.76	18.00	22.76	1.39	0.00	47.36
4	400.00	54.35	24.00	30.35	2.84	0.00	80.55
5	500.00	67.94	30.00	37.94	4.83	0.00	123.32
6	600.00	81.52	36.00	45.52	7.40	0.00	176.24
7	700.00	95.11	42.00	53.11	10.57	0.00	239.92
8	800.00	108.70	48.00	60.70	14.40	0.00	315.02
9	900.00	122.28	54.00	68.28	18.90	0.00	402.20
10	1 000.00	135.87	60.00	75.87	24.13	100.00	402.20
11	1 000.00	135.87	60.00	75.87	24.13	100.00	402.20
12	1 000.00	135.87	60.00	75.87	24.13	100.00	402.20

If no depreciation fund has been established during the development period, so that the company has no income other than that obtained from operation, then, of course, it is impossible to pay dividends at the assumed rate on more than the depreciated value, but the depreciation fund, in that case, is really invested in the equip-

ment, and the stockholders' equity in the same is, and has always been, equal to its depreciated value, a fact that is clearly shown in Table 28. Mr. Nicolaysen.

TABLE 28.—SINKING-FUND DEPRECIATION. NO DEPRECIATION FUND CREATED.

Physical property consists of boats costing $100 each, and having 10 years life. One new boat purchased at the end of each year. Ten boats owned in the tenth and following years. Annual net earnings, $13.587 per boat operated. Stockholders pay in new capital as required and receive 6% dividends. Remainder of earnings invested in new equipment.

Year.	Stockholders' investment at beginning of year.	Net Operating Earnings.			Additional investment by stockholders.	Stockholders' investment at end of year.
		Total.	Return to stockholders.	Invested in equipment.		
1	$100.00	$13.587	$6.00	$7.59	$92.41	$192.41
2	192.41	27.17	11.55	15.62	84.38	276.79
3	276.79	40.76	16.61	24.15	75.85	352.64
4	352.64	54.35	21.16	33.19	66.81	419.45
5	419.45	67.94	25.17	42.77	57.23	476.68
6	476.68	81.52	28.60	52.92	47.08	523.76
7	523.76	95.11	31.43	63.68	36.32	560.08
8	560.08	108.70	33.60	75.10	24.90	584.98
9	584.98	122.28	35.10	87.18	12.82	597.80
10	597.80	135.87	35.87	100.00	0.00	597.80
11	597.80	135.87	35.87	100.00	0.00	597.80
12	597.80	135.87	35.87	100.00	0.00	597.80

It is true that all the current earnings may have been paid out to the stockholders, and these may have paid in annually the full amount needed for additions, but this does not increase their investment any more than it would be decreased if all earnings should have been used for additions and no dividends paid, in which case the capital directly supplied by the stockholders falls short of their actual new investment by an amount equal to a fair return on the money previously invested.

In praxis, it will probably be found, in a majority of cases, that earnings during the first few years of a company's existence, are insufficient to provide for proper depreciation and dividend payments. In such cases the accumulated difference between what may be termed "fair earnings" and actual earnings, should normally be considered as development expenses and included in the intangible values. When this is done, it becomes unnecessary to assume as before that earnings from the very beginning shall be large enough to provide for a fair return on the value of the property. If early earnings are less than fair earnings, the company under consideration will later be entitled

Mr. Nicolaysen.

to a return both on its physical property and its development expenses, but that portion of the return which is based on the physical property will be exactly the same as that previously found.

The assumed rates of return on capital invested and on the depreciation fund were made identical, for the reason that normally it will be found to be both possible and desirable to invest the depreciation fund in additions and betterments. In view of the fact that the element of risk is largely eliminated from regulated utilities, the speaker is inclined to consider the assumed rate—6%—as fair, but it is not his intention to claim that it is necessarily the proper one, the object being primarily to fix a certain rate so that the tables could be calculated.

The claim is often made that if a depreciated plant is able to give as satisfactory service to the public as it did when new, it should also give the same return to the owners. This argument is entirely beyond the speaker's comprehension. If its correctness was recognized, it would seem that the ferry company previously considered could be made a very profitable investment by the simple expedient of buying old, but still serviceable, boats in place of new ones.

The speaker believes that the apparently very great differences between the companies here considered, and actually existing utilities, on investigation, will be found to be unessential.

In all cases the physical property may be divided into three classes: Class I is to include all items of property renewed out of capital; Class II comprises all groups of items which through annual renewals out of earnings are maintained indefinitely in approximately their present condition; and Class III contains all groups of items which eventually will be included in Class II, but which have not yet reached the stage where renewals make up for depreciation. Justice is done, both to the public and to the stockholders, when earnings are large enough to provide for depreciation payments on the sinking-fund basis on all property in Classes I and III, and to yield in addition a fair return to the stockholders on intangible values, on the "value new" of property in Classes I and III and on the depreciated value of property in Class II.

Mr. Lippincott.

J. B. LIPPINCOTT, M. AM. SOC. C. E. (by letter).—The report of this Committee is a most satisfactory and clear-cut discussion of this important subject. If the American Society of Civil Engineers can assist in standardizing methods for estimating reasonable rates for utilities in cities, it will be conferring a benefit both to the investor and to the public. The Committee states (page 21):

"The allowance for overhead charges has generally been substantially under-estimated by commissions and the Courts, as well as by engineers of limited practical experience in construction work. The

latter are prone to assume that unit prices bid by contractors or determined during the progress of the work by the inspector are fair measures of its final cost to the owner, but such is rarely the case." Mr. Lippincott.

In building an engineering work there are two classes of expenses: First, the expense of building the structure itself; and, second, the indirect expenses which are unavoidable and for which there is little to show after the work has been completed. Engineers who have been connected with large enterprises, and have carefully kept segregated cost accounts, are fully aware of the gravity of these indirect charges, whereas others, who have not had this experience, are apt to minimize them. It is to be regretted that so few such cost records are available. Estimates are prone to be below the actual cost. This is not due so much to failure to comprehend the cost of the structure itself, as to lack of realization of these unavoidable auxiliary, incidental, or general expenses. For the same reason, engineers or Courts who are evaluating properties are apt to consider only such structures as they may see on the ground. The worn-out tools and equipment, the monies paid for injuries to men and animals, or police protection, all have left on the ground no visible trace that may be appraised, yet they are clearly an expense incident to the construction.

Any official body whose duty it is to fix a rate for a public utility has a grave responsibility, not only to the public, but also to the investor. Justice requires that each shall be treated with fairness, and, usually, this is the intent. It is necessary for the proper development of a new country that great enterprises should be encouraged. Therefore, particularly if business profits, commensurate with the risk incurred, are not to be allowed in fixing the rates for utilities, and these institutions are not safeguarded against losses, all legitimate expenses incident to the construction should be fully included in the cost of the works, if this cost is to be the basis for fixing rates.

In the construction of the Los Angeles Aqueduct it was possible, to an unusual extent, to learn the real cost of the different classes of a great variety of work, as this structure was built on a day-labor basis instead of by contract, and because an accurate system of cost accounting was maintained. The writer was the Assistant Chief Engineer through its entire survey and construction. The engineer who directed and estimated on the work was in a position to follow closely the reasonableness of all expenses through these cost records. The Aqueduct was finished within the estimated cost; the organization was entirely free from politics, and the monies were all expended without scandal on account of waste or graft.

The methods of accounting used on the Harriman System of railroads were used on the Aqueduct; these provide for all work to be authorized by "engineers' work orders". Every item of expense was

Mr. Lippincott.

charged to some specific "work order". Labor distribution sheets and foreman's material diaries were used. The distribution of accounts was made in the field and checked at headquarters, both by a "Cost Engineer" and by the Auditing Department. The total expenditure and the work order charges were balanced monthly.

Mr. O. E. Clemens, an engineer who was engaged in the field during the early portion of the construction work, showed unusual adaptability to the matter of cost-keeping, and, for this reason, he was transferred to the office of the Chief Engineer and given the title of Cost Engineer, in which position he served during the greater portion of the period in which the Aqueduct was under construction. Subsequent to the completion of the Aqueduct, he made the final general summary and analysis of cost data from which the figures given in this discussion were obtained. The system, as developed by Mr. Clemens, was unusually satisfactory and efficient.

The expenditures were divided between the construction of (1) The Waterway and Its Appurtenances; (2) Auxiliary Construction Expenses and Operation; (3) General Miscellaneous Expense and Operation; and (4) General Office and Executive Expense.

"The Waterway and Its Appurtenances" (1) included the finished structure which is left as a result of the effort. This is mainly the conduit which conveys the water.

"The Auxiliary Construction Expense and Operation" (2) included surveys and general engineering of a preliminary nature, water-pipe lines, for construction and domestic use, telephones, roads and trails, buildings, low-tension power lines, division administration (which included superintendents, division engineers, and office force, material handling, sanitation, and housing), miscellaneous tests, expended cement sacks, patrol of the Aqueduct, miscellaneous losses, reorganization expense, concrete replacements, and net equipment expense after its salvage. This auxiliary expense is the largest indirect charge of this nature, and, for that reason, it may be desirable to give some further analysis of this account. It is divided as shown in Table 29.

The "General Miscellaneous Expense and Operation" (3) included passenger transportation for labor, water investigations, sundry adjustments, subsistence losses, unadjusted freights, stock service losses, and miscellaneous operation adjustments. This account was credited with revenues from the sale of domestic water during construction, rentals, and other incidental revenues.

The "General Office and Executive" (4) account included the salaries of all general officers, accounting, disbursing, purchasing department, and legal expenses, executive railroad transportation, and automobile service. Interest charges or development expenses are not included in any of these items.

TABLE 29.—AUXILIARY CONSTRUCTION EXPENSES AND OPERATION (2). Mr. Lippincott.

Items.	Total cost.	Percentage of waterway cost.	Average cost per foot.
Surveys and general engineering	$366 932	2.30	$0.311
Pipe lines, net	281 645	2.77	0.375
" " M. and O.	162 780		
Telephones, net	75 638	0.95	0.127
" M. and O.	75 115		
Roads and trails	274 395	1.93	0.261
" " " M. and O.	34 237		
Building expense, net	336 531	2.10	0.285
Low-tension power lines	59 240	0.35	0.05
Division administration	775 412	4.86	0.657
Miscellaneous tests	39 167	0.25	0.033
Expended sacks	49 553	0.31	0.042
Patrol of Los Angeles Aqueduct	2 672	1.20	0.163
Miscellaneous losses (not including M. O. H. final)	88 130		
Reorganization	28 580		
Concrete replacements	72 752		
Equipment expense, net	1 548 721	9.20	1.31
Totals	$4 271 500	26.72	$3.614

The summary of costs of the aqueduct construction, exclusive of land purchases and cement plants, and the percentages as compared with cost of waterway, were as follows:

(1) Waterway	$15 942 489	.....
(2) Auxiliary expense	4 271 500	26.72%
(3) Miscellaneous expense	277 038	1.75%
(4) General executive	843 944	5.30%
Total	$21 334 971	

Of this amount 33.77% of the waterway cost was for indirect charges. It may appear at first that these indirect charges were unusually high on this Aqueduct because of the remoteness of the work. However, transportation expenses, whether by rail or wagon, were included in the direct cost of the work. The auxiliary charges were practically as great on the end of the Aqueduct next to the City of Los Angeles as they were on the remote portions of the line.

In a large undertaking, these overhead expenses are proportionately less than on a smaller one. This general average was borne out consistently on the different divisions of the Aqueduct. On those divisions where the total construction costs were low, the percentage of overhead was relatively high, and where the total construction costs were high, the overhead charges were proportionately low. On the Los Angeles Aqueduct, during the greater portion of the work, no passenger

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transportation was paid for labor. As the construction neared completion, however, and the length of time left for finishing the work was becoming short, some passenger transportation for labor had to be paid. The subsistence losses were largely for meals furnished men who never worked any time. These items are included under "Miscellaneous Expense" (3).

The first issue of bonds for the Los Angeles Aqueduct was for $1 500 000, in 1905. This issue bore interest at 4 per cent. This fund was used for the purchase of lands and water rights and for surveys and general investigations. It was followed, in 1907, by an election authorizing the issuance of $23 000 000 in bonds for construction purposes. Of this latter issue, $1 033 600 bore interest at 4% and the remainder at 4½ per cent. The total premium was $77 010, and the bonds were sold as required for funds to carry on the work. The Aqueduct was completed on schedule time in the summer of 1913, and the interest charges on all bonds up to that date (June 1st, 1913) amounted to $3 662 828.43, or 23% of the cost of the waterway. This does not include or take into account the main power-plant construction or interest thereon. This 23% added to the 33.77% for auxiliary expenses actually incurred on the Aqueduct, as previously stated, makes 56.77% total expense for these charges as compared with the actual waterway cost.

On work of a similar character carried on in a less remote region, certain adjustments of these percentages for auxiliary expenses may be made. Roads and trails, buildings and telephones, amounting to 4.98% on the Aqueduct, may be entirely eliminated. The freight on the equipment was a considerable portion of the charges against this item, and the 9.20% may reasonably be reduced to 8 per cent. Other expenses, for replacements, supervision, patrol, etc., should remain the same. We have then for the adjusted Aqueduct auxiliary expenses 26.72% — 6.18% = 20.54 per cent. In the case of work done in a highly developed region, there are more damage claims than on work in a more remote region. The miscellaneous and general executive expenses should not be affected materially. An ordinary work would not involve so long a period of construction, and, consequently, the interest charges would not be so high. However, if consideration be given to the period during which a water-works is being put into full use and earning power, an interest charge of 10% would not be unusual.

Due to the severe employers' liability law that has been passed recently in California, the heavy damages that may be collected by workmen who are injured, irrespective of whether or not they contribute to the accident by their own negligence, introduces an additional new expense in construction work in this State. The ordinary rate for insuring a pay-roll prior to the passing of this new law, against such damage claims, was 1 per cent. The insurance rates at

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this time amount to $5.25 per $100.00 of the pay-roll for laying water mains, $6.12 for erection of pumping stations, dams, and reservoirs, and $12.25 for tunnel work. Owing to the magnitude of the Aqueduct construction, the Board of Public Works did not carry accident insurance, but settled its own damage claims, charging the losses to "Auxiliary Expense and Operation" account. This loss amounted to 0.26% of the pay-roll. However, this work was performed prior to the time when the new employers' liability law went into effect, and settlements were not made under its provisions. Allowing for the present insurance rates for water-works construction, there should be added to the auxiliary expenses for the above, 5% of the labor cost on the work. These labor costs on the Aqueduct were 60% of all costs, and probably amount to at least one-third of the total construction cost on an ordinary water-works. Consequently, 1.67% should be added for liability insurance in California. From this should be deducted the 0.26% already charged to this expense on the Los Angeles Aqueduct, or, say, 1.40% for a net amount.

All the figures in the estimate of cost of this Aqueduct are based on day labor, without profit. A board of four engineers, of which the writer was a member, was employed by the City to determine the value of the Los Angeles Water-Works which the City was purchasing. In this appraisal it was conceded that allowance should be made for contractor's profit, on the assumption that if a new plant was to be built quickly and without any basic organization, to replace the existing plant, the work would be done by contract and not by day labor, and that a reasonable allowance for contractor's profit would be 6 per cent. The Board states: "After due deliberation it is our firm conviction that a further 6% should be added for a (contractor's) profit." The arbitrators slightly increased these estimates, and the final amount paid was higher. On the cost of the completed structure, 6% would be a low allowance for contractor's profit on the construction of a water-works system.

Therefore, the following estimates of auxiliary cost for work similar to that done on the Los Angeles Aqueduct, adjusted to less remote localities, done by contract, and with 10% allowed for interest during construction, are given:

(2)	Auxiliary expenses	20.54 per cent.
(3)	Miscellaneous expense	1.75 "
(4)	General executive	5.30 "
	Interest during construction	10.00 "
	Liability insurance	1.40 "
	Contractor's profit	6.00 "
	Total auxiliary expense	44.99 per cent.

Mr. Lippincott.

These figures are believed to justify an addition of 45% to the direct construction costs of a water-works of the general class previously referred to, to provide for auxiliary expenses.

In addition, a water-works is usually constructed in a fragmentary or piecemeal manner, and not as one comprehensive undertaking, all of which could be put through in a minimum period of time. Work done in small units is more expensive than if built as a whole, vigorously prosecuted, with ample financial backing, as is ordinarily assumed to be the case in making a valuation report. Cheap work must be done with the maximum organization that can be placed on the entire job efficiently, in order to reduce overhead expense. This argument would justify using higher unit cost figures, in making a valuation, than those which are available from a large piece of work that is built under full headway.

The writer believes that we have reached the time when the investors in many of the securities of public utilities need helpful support in the matter of reasonable rate fixing. Irrespective of indiscretions and bad management in the past, we should look forward to reasonable service and fair treatment in the future. If these institutions are not to be guaranteed against loss, and are only to be permitted to earn such reasonable interest on their investment as could be obtained from first-class mortgages or established commercial enterprises, then certainly, in determining the value of the property, due weight should be placed on all legitimate expenditures, including auxiliary costs. If this is not done, it will follow that either the service will deteriorate or the investors will refuse to continue to place funds in institutions of this nature. In the Southwest there is a decided movement on the part of utility corporations to dispose of their properties, even at a sacrifice, in order to terminate the contentious strife with the public. The California Railroad Commission expresses a realization of this situation in its findings in the Palo Alto Gas case:

"This brings me to a consideration of the final question in the case, namely, the amount of return to be allowed the Gas Company on its plant. No fixed percentage applicable to all cases and all classes of utilities can be established by this Commission. Each case must be judged on its own merit. It may be well that a utility in one community would be entitled to one rate of return while a similar concern in another community would be entitled to a different rate. It may be that a large and solidly established utility will not be entitled to as high a return as a smaller utility which is struggling against adverse circumstances. The most that can be said by way of general principles is that the return should be at least the average return which is earned by other classes of business of the same degree of hazard in the same community. The Commission in fixing a rate of return must be liberal, lest too strict a policy result in turning capital to other fields of enterprise. California needs development by public utilities, and this

Mr. Lippincott.

Commission's policy should be a broad and liberal one, so as to encourage capital to develop the State by legitimate public utility enterprises where needed. The Commission should be careful not to permit an inflation of prices in ascertaining the value of the property of a public utility used and useful for the public purpose, but should be liberal in establishing the rate of return on that value. Bearing in mind all the facts of this case as shown by evidence, I find that a rate of return of 8 per cent. on the value of the property of the Palo Alto Gas Company used and useful for the puplic purpose, as fixed herein, is at least a fair and equitable rate of return. If anything, the rate is too high by reason of the fact that the Commission has been more than liberal in establishing the basis of value."*

In certain rate-fixing litigation before the California State Railroad Commission at the City of San Diego, in which that city was made a defendant in the complaint that originated from certain consumers outside of the city who were served from the municipal system, the Engineering Department of the Railroad Commission in its estimates of the value of the property, made the following allowances for overhead or auxiliary charges for the construction of the Otay and Moreno Dams and the Delzura Aqueduct, the total construction amounting to $2 900 000:

Engineering	5	per cent.
Contingencies	7	"
Interest for 3 years	9	"
Legal and general expense	2	"
Liability insurance	3	"
Total	26	per cent.

The writer is informed that these are the highest overhead charges that have been allowed for work of this character by the Engineering Department of the California Railroad Commission. It is fully realized that exception may be taken by some to the classification of the accounts on the Los Angeles Aqueduct, and the relatively high auxiliary expenses shown by this method of classification, but it is believed that the items are given in such detail that others may reclassify them in such a manner as to compare them with their own expenditures and with other distributions that have been made on other work.

Mr. Noble.

ALFRED NOBLE, PAST-PRESIDENT, AM. SOC. C. E.† (by letter).—The writer believes that the subject of valuation is of such great importance that discussion should be continued at future meetings of the Society.

* Opinions and Orders of the Railroad Commission of California, City of Palo Alto *vs.* Palo Alto Gas Co., Vol. 2, pp. 316-317; Opinion of Thelen, Commissioner.

† This discussion was forwarded by Mr. Noble a short time before his death, and, consequently, could not be revised by him.

Mr. Noble. As one of the Committee responsible for this report, he desires to state that it represents the unanimous view of the six members of the Committee participating in its work, after two years of study. It does not follow that its conclusions are correct, and it is hoped that full discussion will disclose errors, if they exist.

The most general objection to the report is in regard to the treatment of depreciation. The Committee's plan provides for income through rates to return to the investor his capital in installments, corresponding with the estimated reduction of value through depreciation. If the investor receives, through rates, a return of the full investment, together with reimbursement of operating, maintenance, and other necessary expenses, and a due allowance for use of capital, risks, and management, it is claimed that he is justly treated and has no further claim on the public. If, instead of diverting to other uses the installments received on account of investment, he reinvests in extensions or replacements, on which fair earnings are allowed, he is receiving fair treatment from the public. If the Committee's plan does not accomplish this, it is faulty. It seems to the writer that the criticisms are in greater part due to misunderstanding of the Committee's recommendations. The Committee regards it as improbable that a large sinking fund will be maintained under any circumstances, because the owner of the property can make the money earn more by other investments; the preferable investment, where practicable, would be for extensions of the plant, if the money were not needed at the time for replacements.

The Equal-Annual-Payment Method has been proposed in order to make the collections from the public as nearly uniform as practicable during the life of the property, the increasing payments on account of return of investment balancing, or nearly balancing, the decreasing payments for use of capital. This is entirely consistent with the idea of a uniform rate for the same service, whether the items of the plant are new or old.

The return to the investor by the Equal-Annual-Payment Method differs from the Sinking-Fund Method, as usually understood, in this, that the sum of the annual returns of capital equals the investment without the inclusion of interest.

The writer's object in presenting this brief discussion is to ask careful examination of the Committee's plan, and to express not only confidence that it is based on justice to all, but a willingness to modify or abandon it, if it can be shown that this confidence is unwarranted.

Mr. Byers. M. L. Byers, M. Am. Soc. C. E.—After considering what has been said on this subject, it seems to the speaker that perhaps the time is opportune to endeavor to emphasize one or two points, and perhaps to ask the Committee for a little further explanation of the practical

application of its theories of valuation for rate-making, having in mind that it is stated (page 6), that "the valuation is for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more." Mr. Byers.

I.—The Courts state, and it seems equitable to assume, that, in the regulation of rates, the property is entitled to a fair return on its fair value.

$$\text{Fair Return} = \text{Fair Value} \times \text{Fair Rate of Return}$$
$$= \text{Output} \times \text{Tariff Rate (or Unit Selling Price)} - \text{Cost}$$

or,

$$\text{Tariff Rate} = \frac{\text{Fair Value} \times \text{Fair Rate of Return} + \text{Cost}}{\text{Output}} \quad \ldots\ldots\ldots (A)$$

As the tariff rate is something that is to apply to future earnings, apparently we must deal with fair value, and fair rate of return of the immediate future, as a practical proposition—what it would probably be for a year, or whatever period would elapse before the next time of revaluation, or readjustment of rates.

In Equation *A* fair value is determined through the process of valuation of the property. It being "the reasonable value of the property at the time it is being used for the public" which must be used, this varies with changes in the prices of labor and materials, land values, additions and betterments carried out, etc., etc.

Fair rate of return is a quantity which varies with the risk of the enterprise and with the value of money. The value of money is constantly fluctuating, and, therefore, for practical purposes, this must be based on the estimated average risk of the industry and the estimated average value of money during a reasonable period of the future. It cannot be determined accurately, but must be estimated by application of experience and judgment to the problem.

Cost is an item which must be estimated. It depends on output, and output will be greater or less in the railroad business, depending on the tariff rate. If the tariff rate is placed so high that it prevents the business from being handled, the output will be reduced, and the unit cost will naturally be increased because of the overhead charge. Cost also includes a number of items, such as, maintenance, cost of operation, etc., including depreciation.

Output is an item which, in the railroad business, depends on the condition of prosperity of the country, the state of the crops, the tariff rate which is adopted in a particular case, etc. If we have rainfall at the proper time, then the output will be increased. Aided by experience, these must be forecasted in determining and estimating output, which is to be applied in the determination of the tariff rates to be used in the future.

Mr. Byers.

It is clear that, in applying this equation to a gas plant or other utility of that sort selling one commodity, it is comparatively easy to solve, even though the various factors seem to be not of the most definite. When it comes to applying it to a railroad, the speaker must confess that he would like the Committee to go into a little detail as to the practical application of its theories, for now Equation (*A*) becomes:

$$\underset{\text{(thousands of them)}}{\text{Tariff Rates}} = \frac{\text{Fair Value} \times \text{Fair Rate of Return} + \text{Cost}}{\underset{\text{(thousands of them)}}{\text{Outputs}}} \ldots (B)$$

There is still only one equation, but there now become thousands of unknown quantities.

Consider a system like the Pennsylvania Railroad, or the New York Central, or the Southern Railway, or any of the large railway systems. There are hundreds and thousands of stations, there are dozens of terminals, each one of which means a different amount of plant applied for the purpose of transportation.

It is common knowledge that, in the passenger business, passengers are carried over quite a number of lines, and are carried various distances. Under present practices, one passenger is charged one rate to-day because he buys a single-trip ticket; another passenger pays a little different rate because he buys a return ticket; still another passenger going between the same points is charged a little different rate because he buys a fifty-trip ticket; still another passenger has a mileage book, and he pays a different rate. Now, one important thing to emphasize in this particular is this: that these passengers have built their homes, many of them at different places, on different lines, because they could travel at those particular rates. They have their investments made.

The freight situation is the same throughout the whole country. Plants all over the United States have been located where they are, because, under the rate system now in existence, those were the economical places to locate those particular plants. Those rates have not been determined on any such basis as this. They have been determined through the influence of many factors.

If it is attempted to substitute for this system of rates which has been developing in this country for 50 years, a new system built on an entirely different basis, what will be the results to the industries which have been economically located on the entirely different present basis? This point is raised simply to emphasize the importance of this question and the danger of throwing the weight of this Society's influence for or against this proposition without due consideration.

It not only affects the railroads' interests, but millions and millions of capital invested in every form of industry.

Mr. Byers.

The bituminous coal traffic furnishes an interesting point of view from which to consider the use of "Valuation for the Purpose of Rate-Making." An enormous quantity of bituminous coal is carried by various railroads to the Great Lakes for transportation across these lakes to points of consumption beyond. Many coal fields, such as the Brazil, the Ohio No. 2, the Ironton, the Pocahontas, the Kanawha, the Fairmont, the Pittsburgh, the Connellsville, etc., supply the coal for this traffic. These coals compete with each other in the Great Lakes markets, commanding prices at such common terminal points, which depend on the relative qualities of the coals and on no other factors. For example: Pittsburgh coal, due to its better quality as a gas coal, commands a price about 10 cents per ton higher than Ohio No. 2 coal. The cost of mining these different coals per ton varies greatly, depending on the thickness of the seam, character of the roof, equipment of the mine, daily output of the mine, wages of the miners, etc. The distance of the coal from the common markets varies by hundreds of miles by air line, and by railroad routes over which it is transported. The cost of transportation to the railroad company varies with the distance, grades, etc. Under the competitive conditions of to-day and of the past the wages of the miners in the different fields, the freight rate per ton (not per ton-mile), and the selling price of the coal at the mines, have been worked out so that the net profits of the industry (market price minus actual cost of mining, transportation, etc.) are divided between the different parties at interest, namely, the miner, the mine owner, the railroad, etc., on the basis which such free competitive condition has established to be equitable.

May it now be asked of the Committee how, in its opinion, it is possible, bearing in mind such complication of service rendered by the railroad, to utilize "Valuation for the Purpose of Rate-Making" and if, in its opinion, under the peculiar methods and conditions which it would seem must be used (if it can be done at all) in the application of valuation to this purpose, it has included in its report all the factors which must enter into a valuation to be used under such unusual circumstances? To one who has had experience in railway operation, but who has not had experience in the facility with which valuation is applied (no doubt) to the simpler provinces of rate-making for gas and similar utilities, it would be a material source of satisfaction if the Committee could feel it to be a part of its duty in the making of a report on "Valuation for the Purpose of Rate-Making" for a railroad, to indicate how such valuation is to be practically applied.

Assume, for example, that valuation is to be applied for the purpose of rate-making to the railroad which hauls Ohio No. 2 coal (as well as other products) to the Lakes, there to meet the competition

Mr. Byers.

from the other coalfields transporting their products over other railroads. In order to determine tariff rate, it is necessary, among other things, to estimate both cost of operation and output; but the tariff rate on the Ohio No. 2 coal, which is arrived at as the result of the application of this valuation data, may be so high as to prevent entirely the movement of this coal; or the rates established through valuation on other lines may prevent the movement of the other coals and give the entire market to the Ohio No. 2 coal. There is nothing said, in the Committee's list of principles, about the necessity for first valuing and establishing rates on all competing lines before attempting to value and establish rates on the line which is the immediate subject of the inquiry. Yet, how else could the Committee be enabled to compute either cost or output for the line transporting Ohio No. 2 coal, and, in the absence of these figures, how could the valuation be used "for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more?"

It would seem that the Committee was well within the bounds of fact when it stated (page 67): "There are several reasons why a physical valuation, taken by itself, furnishes an unsatisfactory basis for determining rates."

II.—In three ways, the Committee has made the discussion of its report unusually difficult.

In the first place, though the application of its recommendations to railroad properties is, at the present time, probably of the maximum interest, practically all its illustrations and applications of principles are to other public service corporations, differing very much in character from railways.

In the second place, although the Committee is dealing with a subject about which there is a very great amount of misapprehension and looseness of use of terms, it does not furnish definitions of the terms which it uses, and these terms, as applied to railway properties, become, many of them, extremely uncertain in their meaning.

Lastly, it frequently states its view without explanation or proof of its correctness, as where it states, as one of the general principles involved in valuation for rate-making (page 6): "It has been well settled by the highest Courts that the owner of such property is entitled to a fair return upon the fair value of the property utilized in or reasonably necessary to the service," and omits the important additional provision that it shall be a "fair return upon the reasonable value of the property at the time it is being used for the public." Another example: On page 67, the statement is made, "Embankments and the slopes of cuts may become more stable on account of the growth of grass and weeds, and may therefore be considered to be more valuable, but such value as this, which is not the result of

expenditure by the corporation, should have no place in the valuation of the property." Why not, as it represents value at the time of public use? Mr. Byers.

On page 69, it is stated:

"It has been held by the Committee that the unfavorably located public service property is not entitled to more than reasonable rates for the service it renders even though the property in question becomes a losing venture; conversely, the Committee believes that the favorably located older property should equitably receive liberal treatment by rate-making bodies."

Applied to railroad property, what does the Committee mean by "unfavorably located public service property" and by "unreasonable rates"? The New York Central and Hudson River Railroad follows the east bank of the Hudson River from Albany through Poughkeepsie to New York, whereas the West Shore Railroad follows the west bank from Albany through Newburgh to Hoboken, and *via* ferry to New York. Presumably, the Committee has in mind that, with reference to the Albany-New York business, the West Shore is an unfavorably located public service property as compared with the New York Central. Does it, however, consider the West Shore to be comparatively unfavorably located with reference to business handled between Albany and Newburgh, or between Newburgh and Hoboken, and would it call the New York Central an "unfavorably located public service property" because it cannot transport between these points as conveniently or economically as the West Shore? Probably without exception, every railroad serves some territories that are not served by any other railroad. How then, can any railroad be said to be an unfavorably located public service property, and if there is no such thing as an unfavorably located public service railroad property, then wherein does the quoted view of the Committee apply?

Old timers relate that, years ago, prior to the construction of certain narow-gauge railroads in the oil fields of Northwestern Pennsylvania, it was necessary to haul boilers and other apparatus for well-drilling plants many miles by teams of horses. When the railroads began to reach these developing territories, they adopted a freight rate for the transportation of boilers just a few dollars below the cost of transporting them by teams. Evidently, even this reduction in cost was a benefit to the shipper. Does the Committee consider such a rate as unreasonable under the circumstances, recalling that the traffic was light and high rates were necessary in order to pay a fair return on the cost of the railroad property? If the Committee does not regard this as an unreasonable basis of rates under the circumstances, does it not follow that, where necessary to earn a fair return on the fair value of a railroad property, any rate which

Mr. Byers.

is low enough to permit the business to be moved by rail as against movement by other forms of transportation is a reasonable rate?

III.—The fundamental law governing the use of capital is that it will flow to the point where, risk considered, the prospect of remuneration for its use is the most enticing, and that it will flow away from any point where the prospect of remuneration, risk considered, is not satisfactory as compared with investment opportunities elsewhere.

Capital invested in fixed property, by reason of its immobility, is more vulnerable to injury than liquid capital. The latter, it has been found, after all sorts of efforts to control it, is absolutely free and can only be enticed, not commanded.

In estimating the risk of an investment in fixed property, capital is guided largely by previous experiences with such investments. If previous experience shows that it has been enticed into such investments by false pretenses and has afterward been robbed of the advantages promised it, the result will naturally be an appreciation of the item of risk and a consequent increase in the rate of remuneration demanded for this type of investment in the future.

If, for example, capital is enticed into the transportation industry by the promise of land grants, bonuses, etc., and, later, finds that efforts are made, whenever opportunity offers, to strip it of the fruits of such grants, gifts, etc., through direction or indirection, the result will be to make capital more wary of and to demand higher returns from investments in such industry in the future.

Always, when capital considers investment in any enterprise, it has two fundamental considerations before it:

A.—What of the conditions of return of this capital proposed to be invested?

B.—What of the remuneration for use during the period of investment?

Return of capital is made in two principal ways:

1.—Direct return, as when a mortgage matures and is paid off.

2.—Indirect return, as when the property is sold. In this class, the chance of appreciation or depreciation in sales value must be reckoned with.

The rate demanded by capital for its use is determined by two principal factors: the current value of money (depending on ratio of supply and demand) and the insurance rate demanded by the intensity of the risk.

It is not greatly material, a common understanding being assumed, how this total rate is made up, the principal point being the size of the total. For example: If it requires a return of 8% to entice capital into the transportation industry, it is immaterial to capital whether the

total amount represented by this 8% return is the result of a straight 8% return on the total amount of the capital invested; or a 10% return on 80% of such capital (the computer amusing himself with the idea that the 20% of capital eliminated is something which he terms depreciation); or a 6% rate on the original capital investment, plus an allowance equal to 2% on such capital, such allowance representing an 8% return on what the computer is pleased to term appreciation. The important point to realize is that, barring deception, unless the computer handles his computations so that the necessary total is reached, capital cannot be attracted into the enterprise. Mr. Byers.

On page 50, of the report, the Committee states:

"The present practice generally gives to the corporation, as to the individual, the so-called unearned increment due to appreciation in land values.

"This is incompatible with the fundamental principle that the corporation is entitled to fair return upon the fair value of its property, because, if the land is valued at the increased price, and the appreciation in value of the land is not included in the accounting, the corporation would receive a sum in addition to a fair return amounting to the appreciation in the value of the land."

On page 52, the Committee states in part:

"Where the valuation cannot be made in this way [that is, by taking the cost of recently purchased land], it is the view of the Committee that land can be valued most correctly on the basis of present price of neighboring land of similar character, augmented by the ratio ordinarily found to obtain in that region between land acquired by public service corporations on one hand, and by private parties on the other."

These two principles appear to conflict; for in the second quotation it is clearly intended that there shall be included in fair value the present value of the land, regardless of the fact that this present value is the result of past appreciation. Presumably, the Committee intends that the statement first quoted shall apply only to further appreciation of land after the date of the valuation. Does not the truth of this proposition, however, depend on the definition which the Committee intends to adopt for "fair rate of return", bearing in mind that it is not the method of computation of the fair rate of return, but the amount of such rate, which is essential in attracting or repelling capital? Most industries include land among the properties owned and used therein, and it seems reasonable to assume that the owners in each case expect to secure, in some form, whatever value accrues from the appreciation thereof. When it is said that a certain corporation earned 12% on its capitalization, this is, in the usual sense of the term, in addition to whatever appreciation in values may have occurred during such period. Is not the Committee injecting a very unusual meaning into its use of terms, and one

Mr. Byers.

better calculated to confuse than to clarify the discussion? Certainly if, in other industries of equal risk, it is possible to earn a certain percentage on actual investment, augmented from time to time by land and other appreciation, then fair promise of similar earning must be extended to induce capital to enter the railroad industry.

It is often to advantage to use an extreme case for the purpose of illustration. Let it be assumed, therefore, that a certain corporation begins business with a capital of $1 000 000, of which practically nothing has been expended for land, although a considerable tract of land has been acquired and is used in the industry. The corporation earns, at the start, 10% on its capitalization of $1 000 000. Twenty years later everything with reference to the business and capital of the corporation is unchanged, except that the actual sales value of its land now amounts to $1 000 000. If the corporation went out of business, a new corporation would be obliged to invest $2 000 000 of capital in order to continue the business, and it certainly would expect to earn a fair rate of return—say, 10%—on such $2 000 000 capitalization. Is it not unjust, therefore, to require the old corporation to earn but 10% on the original outlay instead of on the present value? Moreover, how does the Committee reconcile its position in this matter with the statement of the Federal Supreme Court in San Diego Land and Town Company *v.* National City, also quoted with approval in the Minnesota Rate Case Decision, which reads as follows:

"What the company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used for the public."

Perhaps the Committee intended to convey the meaning that failure to allow unearned increment was intended simply as a practical measure existing only during short intervals and for the purpose of reducing clerical and similar difficulties. This would seem to be borne out by its statement on page 50, as follows:

"Your Committee therefore recommends * * * that the land or other property which appreciates in value be valued at present prices, and that the appreciation in the value of such property be offset against the depreciation in the value of the other property."

If such is the case, it is suggested that the situation be cleared up and that a number of definitions would be of material value in this connection.

IV.—On page 65, the Committee states:

"From the time that a public service commission is authorized to control rates, to fix the amount that the corporation may earn as a depreciation allowance, and to control methods of accounting, it is desirable and proper to take into account the excess or deficiency of earnings."

Mr. Byers.

Does the Committee really consider that, the commission having authority to control rates, the fixing of the depreciation allowance and the control of the methods of accounting are items of consequence having bearing on the propriety of taking into account the excess or deficiency of earnings? Is not the control of rates the one essential factor? Is the Committee to be understood as stating that, in its opinion, in any valuation of the railways of the United States, "it is desirable and proper to take into account the excess or deficiency of earnings" since the appointment of the Interstate Commerce Commission with authority to control rates, and that prior to this date the insufficiency or excess of earnings of railways have "nothing to do with the case", as indicated by its statement (page 65): "It is the view of the Committee that an excess or deficiency of past earnings of an unregulated property should not be considered in making a valuation of the property," or does it take the view expressed (page 9) that, "in the case of a property recently created, with accounts kept properly, the best basis for a valuation is the actual reasonable cost as shown by the accounts," assuming that such a property could be found.

Mr. Stearns.

FREDERIC P. STEARNS, PAST-PRESIDENT, AM. SOC. C. E.—The Committee will not be able to close the discussion of its Progress Report on Valuation for the Purpose of Rate-Making before the latter part of the year, and the speaker thinks it desirable to reply sooner to certain features of the discussions printed in the February and March *Proceedings*, and of the oral discussions at the meetings of March 11th and April 2d, 1914.

One of the most common criticisms of the report of the Committee is that it should not have reported on valuation for the purpose of rate-making.

The Committee has stated in its report that valuations for different purposes necessarily differ in some respects, and it took up first the one branch of the subject indicated by the title of the report. It is rather surprising that there is criticism of this proceeding expressed in so nearly the same terms by so many of those discussing the subject, because the matter has been previously discussed by the Society without such criticism.

For instance, in 1912, C. E. Grunsky, M. Am. Soc. C. E., presented a paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates".* In 1913, William J. Wilgus, M. Am. Soc. C. E., presented a valuable paper on the "Physical Valuation of Railroads",† which begins as follows:

"The Herculean task of valuing the railroads of the United States is of vital importance to the entire nation, whether viewed from

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

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the standpoint of rate regulation or its possible outcome, government ownership."

At several places in this paper and in the discussions which followed, reference is made to valuation for the purpose of rate regulation, and to the difference between valuation for purchase and sale and valuation for rate regulation, it being claimed that value new should be used in connection with rate regulation.

Onward Bates, Past-President, Am. Soc. C. E., in the opening statement of his discussion* of the Committee's report, says:

"The valuation of public utilities for the purpose of rate-making is one of the great economic questions of the present day in the United States."

In the speaker's opinion, the physical valuation is the most important factor entering into the valuation of public utilities for the purpose of rate-making, and this Society includes the men who are best capable of making a physical valuation of such properties.

Mr. Robert H. Whitten, in his comprehensive work, "Valuation of Public Service Corporations", states:

"Official valuations of the property of public service corporations are made for four general purposes: (1) Taxation; (2) Accounting and capitalization; (3) Public purchase; (4) Rate-making."

And his treatise is devoted mainly to valuation for rate-making and public purchase.

The United States Supreme Court has consistently ruled for many years that public utilities are entitled to a fair return on the fair value of their property, and a very important element in the value of a property is its physical value.

A Federal Judge in California, who has had much to do with very important rate cases, stated that the most important fact to be determined is the value of the property.

Many of those who criticize the Committee for making a report on valuation for rate-making remind one of the defendant in the case of the borrowed kettle—first, that it was not borrowed; second, that it was cracked—and suggest that though valuation for the purpose of rate-making may be applicable to other utilities, it is not applicable to railways.

C. W. Hudson, M. Am. Soc. C. E.,† one of the critics of this feature of the Committee's report, goes so far as to hold that a railway is not a public utility, as he says it "appears advisable to the writer to separate the rules and methods for valuing public utility properties from those for valuing railway properties."

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* *Proceedings*, Am. Soc. C. E., for February, 1914, p. 341.

† *Proceedings*, Am. Soc. C. E., for March, 1914, p. 713.

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These criticisms lead the speaker to ask, wherein is there a fundamental difference between the property of railroads and other public utilities, which affects the general principles to be applied in valuation for rate-making? Is not the railroad, as well as any other utility, entitled to a fair return on the fair value of its property? Is it not entitled to an annual allowance for depreciation to cover the lessening value of the various items of perishable property? If its value is to be determined, is it not entitled to have included the overhead charges and development expenses, as much as any other public utility, and is it not proper for this Society to discuss how all these subjects shall be treated to produce the most equitable results?

There are many features, not taken up in the report, which need consideration, such as the effect of competition and of interstate conditions, and as Commissioner Maltbie stated, when discussing this subject, "there is no one yard-stick which will apply to all cases."

Every case is likely to present different conditions, and a public service commission charged with the duty of rate regulation will necessarily be confronted with special problems in different cases, which are likely to modify its action. The fundamental principles are the same in all cases.

If railroads are to be excluded because they are competitive and interstate, why should not all other competitive and interstate properties also be excluded, such as telegraphs, telephones, interurban street railways, pipe lines, expresses, etc.?

The speaker is pleased to note that the fundamental principles which the Committee has presented have not been seriously attacked in any direct manner, although they are ignored in much of the discussion. One of these principles is that the owner of the public utility is entitled to a fair return on the fair value of the property utilized in or reasonably necessary to the service. Another is that in any regulation of rates "there shall be taken into account two distinct features:

"First, the annual return, covering interest and profit, to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment.

"Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

Many have attacked indirectly the second of these general principles, holding that it is enough to earn and lay aside only the sums required for the renewal of plant units when they reach the end of their life. Such allowance as they would provide for depreciation, which they

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sometimes term "deferred renewals", is much less than the annual sum which the Committee believes the corporations are entitled to earn on account of depreciation.

In addition to the fundamental principle that the corporations are entitled to annual returns for the use of and waste of capital, it is obvious that they are entitled to be reimbursed for current expenses, including taxes, and that there are other factors entering into the determination of rates.

The Committee also regards it as fundamental that the rates should be made large enough to be attractive to capital, and expressed the view that "the public may be as injuriously affected by too low as by too high rates".

Several members have criticized the Committee on the ground that it has shown deference to the opinions of the Courts, and one member thought it did not show deference enough. There is no doubt that a Court is fallible, and that, in many matters, it is dependent on the testimony and arguments brought before it, but when an able Court, like the Supreme Court of the United States, composed of men who are able to take a broad view of matters which are presented to them, makes a decision to which it adheres consistently for many years, such decision should be deferred to, for two reasons: First, that it is probably a just decision; second, that it represents the highest law of the country, and a neglect to comply with it in cases coming before the Courts will result in failure.

The decision of the Supreme Court which is especially objected to is that relating to depreciation in the case of Knoxville *vs.* Knoxville Water Company, which, in the judgment of the speaker, is logical and just, and, within the speaker's knowledge, has produced higher returns for a corporation than it had been permitted by a rate-making body to receive prior to that decision.

The decision referred to was given in a rate case, and states clearly that "some substantial allowance for depreciation ought to have been made in this case" from the cost of reproduction. Four years later, in a railroad case, the same Court affirmed the same view by this statement: "and when an estimate of value is made on the basis of reproduction new, the extent of the existing depreciation should be shown and deducted". The decision also says: "It is also to be noted that the depreciation in question is not that which has been overcome by repairs and replacements, but is the actual, existing depreciation in the plant as compared with a new one".

It is better to defer to such well-established rulings of the Court, and to adopt methods of depreciation which will give equitable results under such rulings, than to adopt other methods which will be confiscatory if the Courts continue to rule in accordance with this well-established precedent.

Mr. Stearns.

It seems to the speaker that those so strenuously claiming value new and the smaller annual sum for depreciation which properly goes with it, while the Court decisions are as they are, occupy much the position of a man who wishes to walk to a destination beyond a high stone wall. There are two paths of equal length leading toward this destination: On one the opening has been closed, and on the other the opening is still available. The man would be successful in reaching his destination who goes by the unobstructed path.

C. B. Burdick, M. Am. Soc. C. E.,* apparently thinks that the Committee has not deferred sufficiently to the decisions of the Courts, and asks: "Has the Committee had the opportunity to consider the matter sufficiently to warrant it in recommending procedure at variance with the law as established by the higher Courts?". He adds: "The law is the outgrowth of the experience of centuries".

There are many matters connected with valuation on which the Courts have necessarily acted, but which cannot yet be regarded as well settled. Unprejudiced and intelligent discussion should aid in settling them correctly.

Several have discussed the question as to whether present or original physical conditions should be adopted in making an estimate of the cost of reproduction of an old property. It is recognized that changes are continually occurring, which greatly affect the cost of reproduction of a property, as, for instance, a pavement may be placed over a water pipe which was laid before there was a pavement; or a railroad may be built so as to give access to a dam which was originally built at a long distance from the railroad.

Owing to such changes as these, if present conditions are to be used, the cost of reproducing the water pipe would be largely increased by the cost of taking up and replacing the pavement, and the cost of the dam would be greatly decreased by reason of the diminished cost of providing all supplies and materials under the new conditions.

The Committee has taken the view that the unearned increment, or the unearned decrement, resulting from these changes is not required by any consideration of equity, and that when the estimated cost of reproduction is based on original physical conditions the most equitable results are obtained.

Of those who have discussed the subject, J. E. Gibson, M. Am. Soc. C. E.,† argues that the owner of a public utility is entitled to the unearned increment of value due to highway improvements constructed by the public, but he intimates that a decrement resulting from applying present conditions should not be treated in the same way, as he says:

* *Proceedings*, Am. Soc. C. E., for March, 1914, p. 706.

† *Proceedings*, Am. Soc. C. E., for February, 1914, p. 368.

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"The board of appraisers, in considering the present conditions of the plant, should also consider the conditions existing previous to the commencement of construction; otherwise, the utility might be done a great injustice."

Henry Floy, M. Am. Soc. C. E.,* believes in reproduction new under present conditions.

Allen Hazen, M. Am. Soc. C. E.,† apparently takes the same position as Mr. Floy, as he does not admit that the history of a water pipe "has anything to do with the cost of reproduction of the property as it now stands, or with its value".

Mr. Burdick shows the Committee's position in the matter of present or original conditions quite well when he says:

"It would seem that the Committee has shown a leaning toward investment as a measure of value for the purposes of rate-making, but through the influence of Court decisions, which are numerous and strong, had been forced to the adoption of present prices in the estimates of value".

The use of present prices prevailing at or near the time of valuation is made necessary by Court decisions, and also may be defended on the ground that it is the usual custom in valuing properties. The owner of the utility in this instance does not get an unearned increment or decrement, because he has to take the risk of both increase and decrease in prices. It is not desirable, however, that there should be changes in the valuation of property owing to changes in prices, and this can be avoided in the future with suitable legislation and with continuous commission control, as has been pointed out by the Committee.

In reference to the criticism that the Committee should be consistent and either use original prices with original physical conditions or present prices with present physical conditions, the speaker sees no inconsistency in the Committee's recommendation. The increments and decrements of value due to rise and fall in prices appear to be unavoidable under present conditions, but this does not furnish a reason for introducing increments and decrements due to changing physical conditions, especially as, in many cases, they would be very large.

Mr. Burdick says: "It would seem most logical in the valuation of these properties to pursue one logical method through to the end. If it be the cost of duplication at present, then reproduce under present circumstances", and yet, referring to the reproduction cost of a reservoir, he does not accept the present circumstances under which the railroad passes around the reservoir, but reverts to original conditions and says: "If a railroad formerly occupied the site of the reservoir,

* *Proceedings*, Am. Soc. C. E., for April, 1914, p. 1126.

† *Proceedings*, Am. Soc. C. E., for March, 1914, p. 713.

it would be entirely reasonable to assume that it would necessarily be removed in reproducing at the present time". Mr. Stearns.

The next point to be taken up is the question as to whether value new should be used in all cases where a valuation is made for the purpose of rate-making.

A very large number of those discussing the Committee's report contend that value new should be used in all cases. They do not discriminate, as the Committee has done, and state that value new should be used with some methods of providing for depreciation and not with others, but apparently claim that it should be used in all cases.

George F. Swain, Past-President, Am. Soc. C. E., has presented this view in the most extended and forceful manner, and his discussion will be the basis of the speaker's remarks on the subject. It may seem rather ungracious to select his discussion for criticism in this respect, because he has paid the Committee the high compliment of commenting favorably on all parts of its report except that relating to depreciation, and signifies his agreement with its conclusions as to most of the other features of the report.

In regard to the subject of depreciation, he believes "that the views of the Committee are lacking in clearness, that they involve confusion of thought and are misleading". A comparison of the views of the Committee and of Mr. Swain is, therefore, in order.

First, as to fundamental principles. the Committee, in its report (page 32), takes the view that:

"The corporation, under normal conditions, in addition to an annual return for the use of its capital, is entitled, as already stated, to an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

It also takes the view that the money received by the corporation for depreciation allowances should not be returned to the stockholders as a part of the income of the property, but should be retained as a part of the capital account. In portions of his discussion, Mr. Swain appears to be in entire accord with these views of the Committee. He says:

"The Committee is perfectly right in stating (Addendum, page 3): 'that the owner of a public utility property is entitled to have the rates made high enough, under normal conditions, to pay for the lessening value of the perishable property, so that when such property goes out of use he shall have received, not only a fair return for the use of the capital invested in such property, but shall also have the capital invested in it paid back to him,' except that the original capital should not be paid back to the investor, but should be held by the corporation for renewal when the time comes."

Mr. Stearns.

His exception does not represent a radical difference from the views of the Committee, as it fully agrees with his statement "that the original capital should not be paid back to the investor", and differs in regard to the last part of the exception only in the fact that the Committee would have the capital paid back to the owner of the property reinvested in the property when practicable, without regard to whether the reinvestment is in additions or renewals, and when this course is not practicable held temporarily in a reserve fund or used for retiring outstanding obligations.

In another place he makes this statement, regarding the fundamental principles of depreciation:

"In the speaker's opinion, the method of depreciation to be allowed is an accounting problem, the object being simply to set aside out of earnings each year such sum as may enable all renewals in kind to be made without any increase of capital."

After the endorsement of the statement made by the Committee, this is somewhat "confusing", as it limits the earnings on account of depreciation to the sum which will enable all renewals to be made without increase of capital, but makes no provision for obsolescence.

He refers to the depreciation allowance as being "a payment to the corporation and not to the stockholder who has invested his money", and adds: "It is a payment which should not reach the stockholder, but should be held by the corporation for a specific purpose." This is in entire accord with the views of the Committee.

After this declaration, it is somewhat "confusing" to find the statement that it is not of any consequence to the rate-payers, as such, "what the company does with the depreciation allowances which they have paid. It makes no difference to them whether the company puts these depreciation allowances in a fund * * * or whether it distributes them as dividends"; and then a later statement: "The rate-payer, of course, is concerned that the depreciation funds should be safely invested."

Many rate-payers have found to their sorrow that depreciation funds distributed to the stockholders as dividends are not "safely invested", and that the practice has resulted in unsatisfactory service and in demands for higher rates.

Mr. Swain claims that value new should be used with the Straight-Line or Equal-Annual-Payment Methods of depreciation, as well as with the Sinking-Fund Method.

The speaker will test this view by an example, using the item of property referred to in the Committee's tables on pages 34 and 40 of its report. The assumption is a property having a 20-year life, valued when new at $100, and with computations of depreciation allowances based on 5% interest compounded annually. If such a property is to

be valued new under different methods of depreciation, the annual allowance for depreciation by the Sinking-Fund Method would be $3.02, and by the Straight-Line Method, $5.00. Mr. Stearns

The Sinking-Fund Method used in connection with value new is, in the judgment of the Committee, equitable, as will be seen by reference to page 72 of its report, where it says: *"The Sinking-Fund Method* assumes that returns are to be based on the full value of an item of property." Mr. Swain also supports this view.

If the return for the use of capital is reckoned at 7%, the rate-payers, under the Sinking-Fund Method, would pay $7.00 + $3.02 = $10.02, each year of the life of this item of property for the use and amortization of the capital invested in it. Under the Straight-Line Method, if value new is used, they would be required to pay instead, for the same purposes, $12.00 each year. Is it not "confusing" to claim that $10.02 per year is equitable as a return for these purposes, and that $12.00 is also equitable? If the $10.02 is an equitable amount, as both Mr. Swain and the Committee agree, then the $12.00 represents each year a confiscation of the property of the rate-payers equal to practically 2% of the cost of this item of property.

The Straight-Line Method becomes equitable as between the rate-payer and the corporation only when the valuation of the item of property is reduced from year to year by the amount which the investment in that particular item of property has been repaid by the rate-payers. This does not mean that the investor in the original property will not continue to receive returns on the full amount of his investment, because the part of the investment which is returned to him through the depreciation allowances can be reinvested just as the investor in a note, when a part of the principal has been repaid to him, can reinvest the money so received and continue to receive interest on the full amount of his principal, although not from the holder of the original note.

The inequitable results obtained when using value new with the Straight-Line Method also obtain when using such value with the Equal-Annual-Payment Method, but the amount confiscated from the rate-payer in the latter case is not as large as when the comparison is made with the Straight-Line Method.

Mr. Swain, referring to Table 5 in the Addendum to the report, quotes the statement of the Committee that:

> "It would be grossly unjust to the rate-payers to permit the owner of the property, under the conditions presented, to earn dividends, not only on the $100 000 which he has invested, but also on the additional $117 609 contributed by the rate-payers, and yet this would be the result of using value new in connection with the Equal-Annual-Payment Method of providing for depreciation."

Mr. Stearns.

He then makes this statement:

"The speaker considers it entirely incorrect, with the understanding that the company must renew an item, when worn out, at its own expense without increase of capital. It is entitled to earn a fair return on the items in the last column plus an original depreciation allowance, and at the end of the tenth year, under these circumstances, it will have earned enough to enable it to replace the original plant at its own expense, and will have accumulating funds which will replace the later investments when they require replacement."

An analysis of his statement is in order:

The example given in Table 5 is that of a new plant costing $100 000, having a 10-year life, with the earnings for depreciation invested in additional plant. The total amount of capital invested by the owner from the start to the end of the tenth year is $100 000, but, on account of earnings for depreciation under the Equal-Annual-Payment Method invested in additions to the property, the total value new of the plant increases year by year from $100 000 at the beginning to $217 609 at the end of the ninth year.

Mr. Swain contends that the company is entitled to earn a fair return on the items in the last column—that is, on the value new of all the property, including the part paid for by earnings for depreciation—plus a depreciation allowance.

If, for the purpose of illustration, 7% were to be assumed as a proper annual return for the use of capital, a return of $7 000 each year for such use would meet all the requirements of equity. If, instead of taking 7% of the original investment, the property paid for by contributions from the rate-payers is also included, and all is valued as if new, the 7% per annum would amount to the following sums:

Year.	7% return.
1st	$7 000
2d	7 507
3d	8 085
4th	8 747
5th	9 502
6th	10 365
7th	11 350
8th	12 477
9th	13 763
10th	15 233
11th	16 911
12th	18 339

This tabulation shows that 7% of the value new would give a return of $15 233 in the tenth year, which would be 15.2% on the actual investment by the owner of the property.

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The illustration assumes that additions to the property would be built out of earnings to the extent that the earnings sufficed for this purpose, thus deferring to the same extent the necessity for new capital. As a result, it would be necessary to supply $100 000 of new capital at the end of the tenth year, when, by the assumption, the original property would be replaced, and $7 238 of new capital at the end of the next year, to replace the property built out of earnings at the end of the first year.

Strong objection has been raised by Mr. Swain and others to the suggestion that new capital may be used for renewals.

It would obviously be wrong to use new capital for this purpose if the additions purchased with the rate-payers' contributions for depreciation had been capitalized, as the capital account would be unduly swelled by so doing; but, where the increase of capital has been deferred by paying for additions out of earnings for depreciation, and, as a consequence, new capital is necessary to pay for renewals, the method is not inequitable.

The depreciation allowances invested in additions may represent a depreciation reserve as well as when invested in a fund, and, as a matter of accounting, it is feasible to draw from the depreciation reserve, however it may be invested, for making renewals, but, as a matter of fact and not of accounting, the money thus invested in additions is not available for renewals, but new money must be provided.

Reverting to the table under consideration, it will be seen that if the depreciation allowances invested in additions to the property are considered to be a depreciation reserve, they are sufficient at all times to maintain the investment intact and to pay for the renewal of the property when it reaches the end of its life. Therefore, earnings based on value new, over and above 7% on the amount of the actual investment by the owner of the property, represent a surplus which is not needed for the renewal of the property.

In the first 10 years alone, the accumulated surplus, using 5% compound interest, would amount to $38 308, or about three-eighths of the original investment, and this would be the measure of the confiscation of the property of the rate-payers by using value new in connection with the Equal-Annual-Payment Method of depreciation.

At several places in Mr. Swain's discussion he admits that the combined return for the use of capital and for the depreciation allowance, by the method recommended by the Committee, is fair when the rate of interest used in computing the depreciation allowance is the same as the rate of return for the use of capital. It would be difficult for him to do otherwise than to admit this, because the rates determined by the Committee's Equal-Annual-Payment Method, using the depreciated value of the property as the basis, are identical with

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the rates under corresponding conditions by the Sinking-Fund Method, using value new as the basis.

It is gratifying to have even this admission, because it shows that the use of the depreciated value of property is equitable under some conditions by the Committee's method. This being the case, how can it also be equitable to claim the much higher rates resulting from the use of value new?

Mr. Swain makes the claim that where the rate of interest used in computing the depreciation allowances is less than the rate of return on capital, the property of the stockholder would be confiscated. This claim is not a valid one.

The Committee's plan, as illustrated on page 34 of its report, provides depreciation allowances for amortizing items of property equal to 100% of their value, and a return at the assumed rate on all the investment in the item of property which has not been repaid by the rate-payers. The method, therefore, has incorporated in it all that is necessary to meet the fundamental equitable principles laid down by the Committee.

It is true that Column 7 of the table on page 34 indicates decreasing rates as the property grows older, but it does not follow that decreasing rates are inequitable to the owner of the property. If they were, the Straight-Line Method of depreciation would be especially inequitable, as will be seen by the rapid decrease in rates indicated by Column 7 of the table on page 40.

The fundamental difficulty with Mr. Swain's discussion of this subject seems to be his failure to recognize the difference between sinking-fund payments, which must be invested for the benefit of the sinking fund in order to complete it when an item of property reaches the end of its life, and depreciation allowances, like those provided by the Equal-Annual-Payment and Straight-Line Methods, which are a repayment of a part of the investment in the item of property and can be reinvested where they also will earn income for the benefit of the stockholder. When thus reinvested, whether in a fund or in the property, they keep the investment of the stockholder at all times intact and earning returns. No property of the stockholder is confiscated because of the transfer of a part of his investment from one item of property to another, or temporarily to a fund. The continuous ignoring of the returns from such reinvestment of the money returned as depreciation allowances appears to account for much of his adverse criticism.

Toward the latter part of his discussion, Mr. Swain offers an illustration to "make clear the fact that to take the cost-of-reproduction method, using the value-new-less-depreciation, may, in some cases, mean confiscation". He takes as an illustration a public utility property which is assumed not to need expenditures for renewals for

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the first 10 years, and assumes that the amount earned above operating expenses and taxes varies "from 5% in the first year to 12% in the tenth and subsequent years". It is also assumed "that the depreciation is calculated on the Straight-Line Method, taking a life of 25 years, or 4% per annum". The further assumption is made that the original records of this property have been destroyed and that an appraisal is to be made at the end of the tenth year.

He adds:

"The cost-of-reproduction method is used and shows a cost-of-reproduction-new equal to $1 000 000; but the commission says we must take the depreciated value and the depreciation is about 40%, so that as a basis for earnings, the value of the property is only $600 000. To this we may add the 12% in the renewal fund, or $120 000, making a total of $720 000."

He concludes:

"The value made on the basis of cost-of-reproduction-new-less-depreciation would confiscate the property of the stockholders, which was selling in the markets at $1 200 000 and justified that value, and now becomes worth only about $800 000, or perhaps less. If this is not confiscation, then the speaker does not know what confiscation is."

His Table 24, extended so as to cover the whole 25 years, is reproduced as Table 30.

TABLE 30.

Year of operation.	Percentage on investment earned for distribution to investors and for replacements and renewals.	Percentage paid to investors.	Percentage remaining for replacements and renewals.	Actual depreciation in percentage.
1st	5.00	5.00	0.00	4
2d	5.75	5.75	0.00	4
3d	6.50	6.50	0.00	4
4th	7.25	7.25	0.00	4
5th	8.00	8.00	0.00	4
6th	8.80	8.00	0.80	4
7th	9.60	8.00	1.60	4
8th	10.40	8.00	2.40	4
9th	11.20	8.00	3.20	4
10th	12.00	8.00	4.00	4
11th	12.00	8.00	4.00	4
12th	12.00	8.00	4.00	4
13th	12.00	8.00	4.00	4
14th	12.00	8.00	4.00	4
15th	12.00	8.00	4.00	4
16th	12.00	8.00	4.00	4
17th	12.00	8.00	4.00	4
18th	12.00	8.00	4.00	4
19th	12.00	8.00	4.00	4
20th	12.00	8.00	4.00	4
21st	12.00	8.00	4.00	4
22d	12.00	8.00	4.00	4
23d	12.00	8.00	4.00	4
24th	12.00	8.00	4.00	4
25th	12.00	8.00	4.00	4

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It is "confusing" and "misleading" to have presented the matter in this way, because there is a complete omission of the item of development expense. Mr. Swain evidently believes that this expense should be added in making a valuation, because he gives a general endorsement of the conclusions of the Committee in all matters except those relating to depreciation, and these conclusions included a provision for development expense, and because he has stated in his discussion that "if there should be a lack of a fair return in early years, any such deficiency would be included under cost of developing the business, and in subsequent years a return would be allowed on this also".

According to Mr. Swain's assumptions, 12% is a fair return for the use and amortization of capital, and on this basis the deficiency of earnings in the first year would be 7%, in the second year, 6.25%, and so on, until the ninth year, when the deficiency would be 0.80%, with no deficiency thereafter. The sum of these deficiencies amounts to 35.5% of the original value of the property, and, with accumulations at 5% per annum compounded annually, would amount at the end of the tenth year to 48.6 per cent.

Assuming, as he has done, that the original cost of the property was $1 000 000, and that the reproduction-cost-new would also be $1 000 000, the reproduction-cost-new with the addition of the development expense, therefore, would be $1 486 000.

Of this reproduction cost, $111 000 would be on account of the deficiency in the amount paid to investors, and $375 000 on account of the deficiency in the sum for amortizing the property.

If the property continued to earn 8% on the original cost for distribution to stockholders for the remaining 16 years of its life, and 4% of the original cost as a depreciation allowance, the depreciation fund, including the sum to which the company would be entitled as a development expense under the foregoing assumptions, with interest at 5% compounded annually, would amount at the end of the twenty-fifth year to $1 909 000 to amortize property costing $1 000 000. This is another illustration of the confiscatory character of the Straight-Line Method of depreciation when used in connection with value new—confiscatory of the property of the public.

It is not surprising, in view of such a result, that Mr. Swain states that the Sinking-Fund Method is more favorable to the public. The illustration, however, does not warrant his additional statement that it is equally favorable to the corporation.

If, instead of considering the use of value new, the depreciated value of the property is taken, and, as suggested by Mr. Swain, an appraisal is to be made at the end of the tenth year, the cost of reproduction new is $1 000 000. From this would be deducted the total amount of depreciation by the Straight-Line Method, equal to 40%, giving, as the depreciated value of the property, $600 000. To this

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would be added the 12% in the depreciation fund plus interest, amounting in all to $128 000, and the development expense computed on the basis set forth in the table, amounting at the end of the tenth year to $486 000, making a total of $1 214 000 for a property which originally cost $1 000 000.

This looks very much like confiscation, but it is not the confiscation of the property of the stockholders, as claimed by Mr. Swain.

The enormous development expense in this case is due to two assumptions: First, that such a property is entitled to earn annually 8% of its original cost; second, to the use of the Straight-Line Method of depreciation in connection with value new.

A property of this kind is usually financed for a sum equal to at least one-half of its cost by the issue of bonds, which, in a case like that cited, could be sold on a 6% basis, and, with 8% return on the full value of the property, the stock would pay 10% to the stockholders.

For a 25-year property, if value new is to be used, the Sinking-Fund tables, using a 5% basis, give the proper amount to be earned annually on account of depreciation. This proper amount is 2.1 per cent. The annual depreciation charge of 4%, assumed by Mr. Swain, therefore, represents an excess of 1.9% annually on the full value of the property, either for distribution to the stockholders or for the formation of a surplus. If distributed to the stockholders, where the stock represents one-half of the value of the property, this would mean an addition of 3.8% to the 10%, making 13.8% available for dividends under the proposed assumptions.

If the excessive sums earned for depreciation by using the Straight-Line Method in connection with value new were not distributed to the stockholders, but were retained in a fund, they would amount at the end of the twenty-fifth year to $909 000, corresponding to the surplus already stated.

Mr. Swain's illustration will next be presented in a modified form, so as to make it more rational, and the Equal-Annual-Payment Method of the Committee will be used for computing depreciation.

It does not seem to be in accordance with public policy that the percentage paid to investors should be so large that a property costing $1 000 000 would be "bought and sold between investors on the basis of $1 200 000". Money could be obtained without doubt under such circumstances as those represented by one-half bonds earning 6% and one-half stock earning 8%, making the annual income on the whole value of the property 7%, and this amount will be used in place of the 8% in the previous illustration.

Table 31 covers 10 years, the same as Mr. Swain's Table 24. The percentage earned for the 10 years is copied from his table. The percentage paid to investors is the same in the first 3 years, and then 7%, instead of the larger sum. The percentage remaining for replace-

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ments and renewals amounts in the 10 years to 18.25 in Table 31, as against 12% in Table 24. The total amount of depreciation, using the Equal-Annual-Payment Method on a 5% basis, is 26.37%, in place of the 40% in Table 24. Table 31 shows that there would be a deficiency of earnings in the first 7 years and a surplus in the next 3 years. The development expense, assuming it to be the deficiency of earnings, including 5% interest, would amount to $187 000 at the end of the seventh year.

TABLE 31.

Year of operation.	Percentage on investment earned for distribution to investors and for replacements and renewals.	Percentage paid to investors.	Percentage remaining for replacements and renewals.	Actual depreciation in percentage.
1st	5.00	5.00	0.00	2.10
2d	5.75	5.75	0.00	2.20
3d	6.50	6.50	0.00	2.31
4th	7.25	7.00	0.25	2.43
5th	8.00	7.00	1.00	2.55
6th	8.80	7.00	1.80	2.67
7th	9.60	7.00	2.60	2.81
8th	10.40	7.00	3.40	2.95
9th	11.20	7.00	4.20	3.10
10th	12.00	7.00	5.00	3.25
Total for 10-year period.	84.50	66.25	18.25	26.37

If a valuation were to be made, as assumed by Mr. Swain, at the end of the tenth year, the value new would be, as he has stated, $1 000 000. The total depreciation to the end of the tenth year would be $264 000, leaving as the depreciated value of the property $736 000. To this should be added the 18.25% in the depreciation reserve plus interest, amounting in all to $200 000, and the development expense, assuming it to be the same as at the end of the seventh year, $187 000, making a total of $1 123 000 for a property which originally cost $1 000 000.

The adoption in this case of a more reasonable rate of return and a more reasonable depreciation allowance results in a more reasonable development expense and a more reasonable reproduction cost of the property. The reproduction cost, however, is still $123 000 more than the original cost, although so large an excess would not be warranted by the deficiency of dividends during the early years of the life of the property. There is still a confiscation of the property of the ratepayers, and this is due mainly to the assumption that during the first 10 years the earnings are based on value new and not on the depreciated value of the property, as they should be when this method of depreciation is used.

If an illustration had been presented in which the earnings were based on the depreciated value of the property, instead of on value new,

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and if 7% compound interest were used in computing the depreciation allowance, the development expense and the accumulation of the depreciation reserve, the reproduction value of the property, including the depreciation reserve and the development expense at the end of the tenth year, would have been only enough more than $1 000 000 to compensate for the net deficiency of dividends during the 10-year period. Under such circumstances, there would be no confiscation of the property of either the corporation or the rate-payer.

The Committee holds the view that the appreciation of land, and of some other items of property which appreciate in value, is closely related to the depreciation in the value of the structural property, and that, if the public makes good to the corporation the loss of value of the portions of the property which in the course of events are practically sure to depreciate, the corporation should also make good to the public the increase in value of the portions of the property which are practically sure to appreciate.

Mr. Swain takes the ground that an appreciation in land is an appreciation in price. He mentions two kinds of depreciation, one due to wear and tear and approaching obsolescence and the other which may result from the smaller cost of performing work, and states that the appreciation in land should be compared with the depreciation due to the smaller cost of performing work and not to the depreciation caused by wear and tear and approaching obsolescence.

Under existing laws it seems to be necessary to value property at substantially present prices, and the increase or decrease in value, due to changes in the cost of performing work, which, in recent years, has been almost wholly increase, is a risk which the owner of a property under existing conditions is obliged to assume.

In order to obtain equitable results, the cause of a continuing depreciation or appreciation in the value of property is not important. It is the fact that structural property is bound to depreciate which makes it equitable for the public to make good to the corporation annually the amount of such depreciation, and, similarly, the fact that land under most circumstances is bound to appreciate in value which makes it equitable for the corporation to make good to the public the amount of such appreciation.

Owing to a rise in prices, items of property which in time are bound to go out of use owing to wear and tear or obsolescence may be worth more after they have attained a certain age than when new, and, similarly, owing to change in prices, there may be years in which land will not appreciate, but, in the long run, land in a growing community does appreciate in value.

Mr. Swain takes the ground that in appraisals the present value of the property must be taken, and that, if the reproduction cost at present prices means an increase in the value of the property, the corporation is entitled to the increment.

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In this argument, however, he does not take the present value of the perishable property, but the value of such property as it would be if it were new, although it is obvious that an old locomotive, car, or tie has much less real value than a new one. If he were to take the real value of such perishable property, instead of its value new, the same line of reasoning, by which he attempts to show that the corporation should not be required to make good to the public the amount of the appreciation in land, might be used to show that the public should not be required to make good to the corporation the depreciation in the value of the perishable property.

The most common argument against the Committee's position that equity requires the owner of a public service property to account to the public for the appreciation of land is expressed by C. P. Howard,* M. Am. Soc. C. E. He states that "All other owners of property enjoy this increase as a clear bonus; the unearned increment". Is this a correct statement?

In the case of a public utility, the owner should be permitted to charge rates to reimburse the sums paid for taxes on the land, and, under the views expressed by the Committee, he is entitled to have a fair return on the fair value of the land as well as on the fair value of any other property.

The private land owner does not have his taxes paid for him and is not entitled to receive from the public a fair annual return on the value of his property. In most cases where the land is vacant he does not receive more than a meager return, if any, and has to depend mainly on the appreciation in the value of the land in order to obtain finally a fair return.

If it is correct to infer that because private owners enjoy an increase in the value of land, the owner of a public utility should likewise enjoy such increase, is it not equally correct to infer that if the private owner of a building on the land has to stand the loss due to the decrease in value of the building as it grows older, the owner of the public utility should likewise stand the loss due to the decrease in value of a building owned by him, and not claim that the public should pay him a depreciation allowance to compensate him for such loss?

To revert once more to Mr. Swain's discussion, he claims that in the case where the cost of constructing a property is known, the cost new is an admittedly correct method, and that as the cost-of-reproduction-new is a substitute for cost new, when the latter cannot be ascertained, it also should be used instead of the depreciated value.

If cost new were an admittedly correct method, then the deduction made by Mr. Swain would be logical, but, in the speaker's opinion, it is not a correct method when a method of providing for depreciation is used which returns to the owner of the property the full value of each item during its life.

* *Proceedings*, Am. Soc. C. E., for February, 1914, p. 387.

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PAPERS AND DISCUSSIONS

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STEEL STRESSES IN FLAT SLABS.

Discussion.*

By Messrs. L. J. Mensch, W. K. Hatt, and H. T. Eddy.†

Mr. Mensch.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—This paper appears to the writer to be a defense of, and a propaganda for, the so-called mushroom type of flat floor construction.

The author seems to be quite satisfied that mushroom floors with a notoriously deficient quantity of steel reinforcement are safe and proper constructions. This is a debatable question, with reference to the factor of safety required. In some constructions there is a factor of safety of a little more than 1, as in foundations, retaining walls, dams, bridge abutments, etc., in fact, where the carrying capacity and cohesion of the ground is to be considered. In other cases a factor of safety of hardly 2 is often, knowingly or unknowingly, adopted, as in columns, whether of steel, brick, or concrete, especially in outside columns and those which are eccentrically loaded, also in roof trusses, common brick walls in buildings, etc.

The numerous cases where such low factors of safety are found may blind an engineer and make him believe that he is as well justified in adopting a similar factor of safety in flat slab construction. On the other hand, engineers are accustomed to allow a factor of safety of 3 in steel-girder constructions, and the general public demands a factor of safety of at least 4 in reinforced concrete construction, on account of its novelty and its greater uncertainty in erection.

The writer claims that a large percentage of the examples cited by the author have a factor of safety of a trifle more or less than 2, and that his formulas simply accommodate the extensometer tests of these slabs. Extensometer tests do not give any indication of the real strength of a slab, which is reinforced by light bars and to such low amounts as ¼ per cent. The extensometer gives the elongation in a

* Continued from March, 1914, *Proceedings*.

† Author's closure.

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length of generally 8 in. of steel bars embedded in concrete. Where the surrounding concrete is not cracked in the 8 in., the extension obtained by the extensometer, in inches per inch of length, multiplied by the modulus of elasticity of the steel, generally taken at 30 000 000, will give the steel fiber stress, but no correct idea of the interior resisting moment, because the tensile strength of the concrete is disregarded. Assume, however, that there is a crack in the concrete between the contact points of the extensometer, then the stress is transferred by shear and by the bond of concrete and steel to the steel bar in the neighborhood of the crack, and the bar is stressed considerably higher at the crack than at points farther from it.

For an illustration, assume that the reinforcement consists of $\frac{3}{4}$-in. round bars, 6 in. from center to center, that the crack in the concrete is 1 in. deep, and that the tensile strength of the concrete is 200 lb. per sq. in. The surface of a $\frac{3}{4}$-in. bar per lin. in. is 1.18 sq. in., and its cross-section is 0.11 sq. in. The original strength of a concrete section, 6 in. wide and 1 in. deep, is $6 \times 200 = 1\,200$ lb., and, assuming a bond stress of 400 lb. per sq. in., or 472 lb. per lin. in. of $\frac{3}{4}$-in. bar, it will require a length of 3 in. on each side of the crack to transmit the original strength of the cracked portion of the concrete into the bar. This additional stress in the steel fibers amounts to $1\,200 \div 0.11 = 10\,900$ lb. per sq. in., and, for determining the elongation of the bar we have to assume that it acts only on an average of 3 in., though the extensometer measures on 8 in.; hence the stresses obtained by extensometer readings are in this particular case only three-eighths of the actual stresses. In other words, when one tries to judge, from extensometer readings, the distribution of the exterior bending moments, or their equivalent, the interior resisting moments, one is likely to under-estimate the value of the exterior bending moments 50% and more, due to the neglect of the tensile stresses of the concrete.

The fact that extensometer readings do not permit any correct conclusions as to the interior resisting moment, or the factor of safety of the slab, is indisputably proved, by tests made by Richard L. Humphrey, M. Am. Soc. C. E.* In that investigation there were 336 beams, 13 ft. long, 8 by 11 in. in cross-section, and they were tested on a 12-ft. span by two equal loads at the one-third points. The reinforcement throughout consisted of $\frac{1}{2}$-in. bars, having a yield point of approximately 40 000 lb. The number of bars varied from two to eight, corresponding to a percentage of $\frac{1}{2}$ to 2. The mixture of the concrete was 1:2:4 by volume, and the tests were made when the beams were 4, 13, 26, and 52 weeks old. In every beam tested there were careful readings of the deformations, both of the top fibers and of the steel fibers, for stages of loadings of from 500 to 1 000 lb. up to the ultimate load.

* Technologic Paper No. 2, U. S. Bureau of Standards.

TABLE 14.—LOG SHEETS OF GRAVEL CONCRETE BEAMS, 13 WEEKS OLD. Mr. Mensch.

Applied load, in pounds.	$\frac{M}{bd^2}$, in pounds per square inch.	Deformation, in Millionths of an Inch per Inch.		K, percentage.	Deflection, in inches.
		Upper fiber.	Steel fiber.		
Beam 337; ½% Reinforcement.					
0	24.85	17	21	44.6	0.010
1 000	54.85	48	44	52.0	0.020
2 000	84.85	66	63	51.3	0.030
3 000	114.85	112	115	49.4	0.050
3 500	129.85	138	156	46.9	0.060
4 000	144.85	172	236	42.2	0.080
4 500	159.85	237	472	33.5	0.130
5 000	174.85	282	639	30.6	0.170
5 500	189.85	319	815	28.1	0.215
6 000	204.85	359	993	26.5	0.270
6 500	219.85	398	1 162	25.5	0.310
6 740	227.05	.....	1 241	.....	0.340
5 600	192.85	491	1 154	29.9	0.340
Beam 338; ½% Reinforcement.					
0	24.64	21	19	52.5	0.010
1 000	54.64	51	43	54.1	0.025
2 000	84.64	75	63	54.3	0.035
3 000	114.64	115	109	51.4	0.055
3 500	129.64	142	140	50.4	0.060
4 000	144.64	182	217	45.6	0.085
4 500	159.64	235	373	38.6	0.120
5 000	174.64	286	566	33.5	0.170
5 500	189.64	337	797	29.7	0.225
.....	.....	.....	.....	.....	.....
5 000	174.64	411	1 684	19.6	0.330
Beam 348; ¾% Reinforcement.					
0	25.26	23	15	60.4	0.010
1 000	55.26	53	38	58.0	0.025
2 000	85.26	77	56	57.8	0.035
3 000	115.26	113	85	57.0	0.050
3 500	130.26	126	103	55.1	0.055
4 000	145.26	146	127	53.4	0.060
4 500	160.26	167	168	49.8	0.070
5 000	175.26	209	253	45.3	0.100
5 500	190.26	257	362	41.5	0.120
6 000	205.26	293	472	38.3	0.150
7 000	235.26	357	656	35.2	0.210
8 000	265.26	421	865	32.8	0.270
9 000	295.26	481	1 072	31.0	0.330
9 370	306.36	.....	1 154	.....	0.355
9 620	313.86	.....	1 338	.....	0.390
8 000	265.26	534	1 726	23.7	0.395

Applied load, in pounds.	$\frac{M}{bd^2}$, in pounds per square inch.	Deformation, in Millionths of an Inch per Inch.		K, percentage.	Deflection, in inches.
		Upper fiber.	Steel fiber.		
Beam 349; ¾% Reinforcement.					
0	25.47	13	15	47.2	0.010
1 000	55.47	42	39	51.9	0.020
2 000	85.47	64	60	51.7	0.040
3 000	115.47	93	84	52.5	0.050
4 000	145.47	117	111	51.4	0.065
4 500	160.47	145	149	49.3	0.080
5 000	175.47	171	207	45.2	0.100
5 500	190.47	217	321	40.3	0.120
6 000	205.47	257	456	36.0	0.150
7 000	235.47	321	706	31.3	0.220
8 000	265.47	380	913	29.4	0.280
9 000	295.47	433	1 087	28.5	0.330
9 500	310.47	464	1 197	27.9	0.360
10 000	325.47	489	1 294	27.4	0.390
10 500	340.47	516	1 378	27.2	0.415
11 000	355.47	551	1 467	27.3	0.440
11 050	356.97	.....	1 516	.....	0.465
9 300	304.47	548	1 981	21.7	0.475
Beam 350; ¾% Reinforcement.					
0	25.06	19	21	47.5	0.005
1 000	55.06	43	46	48.6	0.020
2 000	85.06	70	68	49.6	0.030
3 000	115.06	102	96	51.5	0.040
4 000	145.06	133	130	50.6	0.050
4 500	160.06	160	169	48.7	0.060
5 000	175.06	198	248	44.4	0.080
5 500	190.06	244	390	38.5	0.115
6 000	205.06	289	533	35.2	0.150
7 000	235.06	360	754	32.3	0.210
8 000	265.06	418	945	30.7	0.265
8 800	289.06	.....	1 250	.....	0.345
7 500	250.06	498	1 597	23.8	0.355
Beam 360; 1% Reinforcement.					
0	25.26	23	17	57.5	0.010
1 000	55.26	56	43	56.5	0.015
2 000	85.26	84	65	56.5	0.025
3 000	115.26	107	87	55.1	0.035
4 000	145.26	135	111	54.9	0.045
4 500	160.26	158	130	54.9	0.050
5 000	175.26	181	162	52.8	0.060
5 500	190.26	206	222	48.2	0.080
6 000	205.26	246	330	42.7	0.100
7 000	235.26	309	508	37.8	0.150
8 000	265.26	358	643	35.8	0.190
9 000	295.26	409	786	34.2	0.230
10 000	325.26	455	938	32.6	0.275
11 000	355.26	508	1 068	32.2	0.320
12 000	385.26	556	1 200	31.7	0.360
12 850	410.76	.....	1 362	.....	0.400
10 700	346.26	626	1 921	24.6	0.430

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In Table 14, taken from Technologic Paper No. 2, it will be noticed that Beam No. 337 failed at a total load of 6 740 lb. At a load of 3 500 lb., which is more than one-half of the ultimate load, the extensometer reading of the steel fiber was $\frac{156}{1\,000\,000}$ in. per in.; or, multiplying this extension by 30 000 000, we obtain a stress of only 4 680 lb. It is well to keep in mind that the extensometer reading gives such a low stress when the factor of safety is less than 2. Similarily, we find that at a load of 4 500 lb., which is two-thirds of the ultimate load, the stress in the steel fibers is 14 160 lb. At 5 000 lb., which is 75% of the ultimate load, the steel fiber stress is only 19 000 lb. per sq. in. Similar conditions may be found in every beam tested, even in those with 2% of reinforcement. One may reasonably expect that the discrepancies will be still greater in flat slab construction, where the percentage of reinforcement is considerably lower than in Beam No. 337, being mostly only $\frac{1}{4}$ to $\frac{1}{3}$ of 1 per cent. One can even imagine the case where, with an insufficient quantity of reinforcement, the slab will break at practically the same load as a non-reinforced slab, where, just before breaking the extensometer would not show a larger extension that 0.0001 in. per in., corresponding to a stress at failure of only 3 000 lb. per sq. in.

In studying the tables it is found that at certain stages a very small increase of the load causes a great change in the extensometer readings, which explains the great discrepancies found by Mr. Eddy in the stresses of adjoining bars or of symmetrical points. The unreliability of extensometer readings for the determination of the interior resisting moment of reinforced concrete beams was discussed by the writer before the Western Society of Engineers in the spring of 1904.*

There is yet another simple indication that the flat slab constructions cited by Mr. Eddy have not the conventional factor of safety, and that is the appearance of cracks at the most dangerous sections, where one would expect to find them at loadings of from one-third to one-half of the ultimate load. From tests of beams reinforced by $\frac{1}{4}$%, it is known that cracks appear at a load which is about three-fourths of the ultimate; and, in slabs reinforced by $\frac{1}{2}$%, at about one-half of the ultimate load.

Mr. Eddy writes:

"Indeed, slab action in general may be described partly as the attempted mechanical superposition of one set of parallel depressions and elevations on another set of similar corrugations at right angle to them. Such sets mutually support each other and give rise to slab action. * * *"

This sounds mysterious, and will not enlighten any one; it only shows that the author does not care to express in a scientific way the

* The *Journal* of the Western Society of Engineers, for 1904, contains a number of tables and diagrams.

real slab action, as has been done, for example, in a masterful way, by the Italian engineer, Danusso,* and by others. Mr. Mensch.

There is no mystery about slab action. It is simply a combination of continuous beams acting in at least two directions, which beams are connected with each other and with the columns, and the deflection in any point is common to at least two beams acting at right angles to each other.

On account of the large sizes of the columns and column heads, and the depression of the floors at the columns, the beams directly connecting the columns, and corresponding to the sides of the squares, have considerably greater stiffness than those parallel to them and nearer the center of the slab, and these beams have a comparatively still greater stiffness than any diagonal beams, with the result that the mechanical action of the whole slab must be nearly identical with the action of a flat slab of a smaller span than the distance from center to center of columns, supported by four wide girders of a depth somewhat greater than the thickness of the slab.

If we omit to take into account the presence of the large columns and column heads, we obtain the case considered by Winkler and Grashof, who assumed the supports of the slab to consist of ideal points, and who found that the bending moment in the center of the span is $\frac{WL}{48}$ and over the columns and side of the square is $\frac{WL}{24}$, when W is the total load on the panel, L is the distance from center to center of columns, and Poisson's ratio is 0.1. The bending moments change only very slightly if other values of Poisson's ratio are assumed. For steel, it is generally found at 0.3, and the moments become $\frac{WL}{26.4}$ and $\frac{WL}{52.8}$, respectively. Returning again to the flat slab as a combination of four wide beams supporting a smaller flat slab, it is only within reason to assume that the wide girders have to carry the same loads as ordinary beams in the case of a flat slab supported by narrow and deep beams.

It is here that one finds a riotous license of figuring by most of the advocates of flat slab constructions. According to most building ordinances and accepted good practice, the bending moments in beams supporting flat slabs must be taken both over the supports and in the center of the beams at not less than $\frac{W}{2} \times \frac{L}{12} \times \frac{3}{2}$ (the latter factor on account of the triangular application of the load over the beam) $= \frac{WL}{16}$, although theory shows that the bending moment in the center of the span is only $\frac{WL}{32}$.

* "Il Cemento," 1912.

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Now, the advocates of the mushroom system are not satisfied to figure the bending moment in the center of the side beams as the theoretical moment, which is only one-half of the moment any engineer is allowed to figure when he adopts common girder construction; they go still further, and claim the right (according to Mr. Eddy's Equation 34) to figure this moment as $\frac{WL}{175}$, which is one-tenth of the moment required by the building ordinances. There we hear again that they must be right or their constructions would fail at the first application of the design load, and that they are right because the extensometer readings bear them out. The writer has shown how erroneous are the results obtained by extensometer readings; it remains yet to prove that the real bending moments in the center of the side beams are less than are required by the building ordinances, otherwise their structures actually would not stand up.

In flat slabs supported by continuous girders of slightly greater stiffness than the slab, the Italian engineer, Danusso, has shown that the load of the slab is distributed nearly uniformly over the girders, instead of with the triangular distribution found for very stiff beams; hence the bending moments for a beam supporting two adjoining panels are theoretically $\frac{WL}{24}$ and $\frac{WL}{48}$, at the supports and the center, respectively. Engineers need not be surprised that the bending moments for the stiffer side beams are just as large as those given by Grashof for the whole width of the slab. This case happens very often in ordinary girder and joist construction. Assume, for example, a flat slab supported by parallel walls, 20 ft. from center to center. If this distance is spanned by a simple slab, the bending moment is $\frac{WL}{8}$. As a rule, it is cheaper to provide girders of 20 ft. span and, say, 10 ft. from center to center, which girders must be figured for a moment of $\frac{WL}{8}$, and the slabs between the girders must be figured for another bending moment. Hence the sum of all bending moments is larger than the original moment $\frac{WL}{8}$, yet the cost of the construction is generally less.

In continuous beams which are of larger moment of inertia near the supports than in the center, such as is the case with the large columns and column heads used, it is known, also, that the moments at the supports become larger and those in the center of the span become smaller; taking also into consideration the fact that the span of the side beams may be safely diminished by one-half of the diameter of the columns, the moments of $\frac{WL}{24}$ and $\frac{WL}{48}$ become in most cases

cited by Mr. Eddy, where no depression in the floor is used, $\frac{WL}{24.4}$ and $\frac{WL}{62}$; and, where a depression is used, they become $\frac{WL}{22.2}$ and $\frac{WL}{92}$. Hence, in the most favorable case, giving the mushroom system all the advantages of continuous action, such as is not adopted by any responsible designer in any other class of work, neglecting at the same time all possible settlements, Mr. Eddy advises the use of a bending moment of one-half of what it can possibly be, which readily explains the low factor of safety. Mr. Mensch.

The advocates of the mushroom system may claim that the diagonal beams help to support a great portion of the load which the side beams are assumed to carry. Mr. Eddy, however, admits that the stresses in the center of the diagonal beams are generally found to be one-half as great as those in the side beams; this, together with the fact that the span of the diagonal beams is 1.41 times that of the side beams, permits the conclusion that a diagonal beam supports only one-quarter of the load of a side beam, and, in fact, simply transmits the loads from the interior of the panel to the side beams.

Mr. Eddy advises calculating the diagonal beams in the center for a moment of about $\frac{WL}{250}$, which does not seem to be sufficient, even for the simple transmission of the loads to the side beams. Granting that the width of the side beams may be assumed to be four-tenths of the span from center to center of columns, the width of the interior slab is six-tenths of L and the bending moment per linear foot of the interior slab, according to most building ordinances, must be figured as $\frac{w\left(\frac{6}{10}L\right)^2}{24} = \frac{wL^2}{66.67}$, while $\frac{WL}{250}$ per diagonal beam corresponds to $\frac{wL^2}{150}$ per lin. ft. of the interior slab.

It is remarkable that Mr. Eddy overlooked the fact that the distance between the points of inflection on the sides of the squares of several of the tests cited by him were found to be from $0.5L$ to $0.55L$. According to his own statements, the bending moment must be larger in the center of the sides than $\frac{w.(0.5\,L)^2}{8}$ and $\frac{w.(0.55\,L)^2}{8}$, or $\frac{w.L^2}{32}$ and $\frac{w.L^2}{27}$, respectively, while his moment of $\frac{W.L^2}{175}$ corresponds to $\frac{w.L^2}{70}$.

The advocates of many flat slab systems declare that the common beam theory does not apply to flat slabs. The writer has never seen a similar statement in the works of St. Venant, Winkler, Grashof,

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Föppl, or Müller-Breslau, who, in much more difficult problems than this, applied only the common beam theory, and in particular, the case of flat slabs was solved by Winkler and Grashof by that theory. There is yet one defense open to the advocates of the mushroom system, and that is the mystic influence of Poisson's ratio, but Mr. Eddy himself states that, on account of bending moments of opposite signs acting at right angles at the center of the side beams, the stresses there are larger than in a common beam; hence he can use it only for a defense of the low moments he assumes over the columns, and he claims that Poisson's ratio must be assumed as one-half. This, however, is a mechanical impossibility, clearly excluded by Grashof, as a simple change in shape without any change in the distance of the molecules is identical with a Poisson's ratio of one-half. Over the columns we have the most unfavorable combination of maximum bending moment and maximum shear acting together in the same section, and Grashof advised, even in the case of steel, reducing the stresses over the columns by 20 per cent. Besides, near the ultimate load, the concrete is cracked in every direction around the columns, and the steel bars will hardly be benefited by stresses acting in two directions, although the concrete in compression may be helped considerably. The percentage of reinforcement being low, an increase of the compressive stress of the concrete of 50% will benefit the internal resisting moment less than 5 per cent.

Mr. Eddy considers only the case of all panels loaded, which is a more favorable one for the center of all beams than that of a single panel, or two panels loaded, and extensometer tests clearly bear this out. In the discussion of a former paper on this subject,* the writer has shown that in the latter case the slab and the supporting columns and the columns above the slab form an arch construction in which the columns are very highly strained by the negative moments at the supports, and that the negative moments in the adjoining panels are very small when the columns have a larger moment of inertia than the slab. These conclusions were proved indisputably by the tests recently made by Mr. W. A. Slater, of the University of Illinois, on a new factory building of the Shredded Wheat Company, at Niagara Falls, N. Y. Mr. Slater presented the results of the test in a paper before the meeting of the American Concrete Institute in Chicago, in February, 1914. Although the floors were tested to only 1½ times the total dead and live load, numerous fine cracks appeared, and the extensometer indicated steel stresses of 15 000 lb. per sq. in. in the slab and additional compressive stresses in the columns, due to the loading of panels on one side of the columns, of 400 lb. per sq. in.

Mr. Eddy declares that the drops in the floors are unsightly, bulky,

* *Proceedings*, Am. Soc. C. E., for August, 1913.

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and unnecessary, and prefers the mushroom type, where the depression of the floor is omitted and replaced by a magic steel ring in the neutral axis of the slab. That this advocates very poor construction, not countenanced by any reputable engineer, may be seen from the detail of the slab of the Northwestern Glass Company's Building, cited by Mr. Eddy. The column caps scale 36 in. in diameter, the dead weight of the floor is 120 lb. per sq. ft., the live load is 400 lb. per sq. ft., and the total load of a panel supported by one column is $16 \times 17 \times 520 = 141\,440$ lb. The floor slab is 8 in. thick, and the sectional area of the slab in shear around the cap equals $36 \times 3.14 \times 8 = 904$ sq. in., or a shear of $141\,440 \div 904 = 155$ lb. per sq. in. of the total section, or of about 220 lb. per sq. in. for a section of the depth of *jd*. Most building ordinances allow only a shear of 125 lb. per sq. in. on the section of a depth of *jd*, and prescribe the use of bent-up bars and stirrups at the section of great shear; on the other hand, the advocates of the mushroom system and their imitators take another license to step over ordinances and sober reasoning, allow a high shear, and no scientific reinforcement for shear whatsoever.

When called to task for allowing such a high shearing stress, these experts defend it by saying that only "punching shear" is acting. The writer does not know that "punching shear" is a term common in engineering, but, if it means anything, it must mean pure shear, like that in a true punching operation, when practically no bending stresses are acting on the material which is being punched. We have shown before that around the columns the maximum moment and the maximum shear are acting together, and that the stresses must be reduced and not increased at that section.

A great many thoughtless and entirely erroneous statements in reference to flat slabs may be found in most articles published on this subject in current engineering literature, and an example is Table 15.

TABLE 15.

No.	Method.	Thickness of slab, in inches.	Steel per panel, in pounds.
1	Cantilever	8	2 189
2	Turneaure and Maurer	12	1 931
3	Grashof	8	784
4	Mensch	8	2 120
5	Turner	8	549
6	McMillan	8	1 084
7	Brayton	8½	1 900

This table is from a discussion by A. W. Buel, M. Am. Soc. C. E., in *Proceedings*, Am. Soc. C. E., for August, 1913.

From Table 15 it would appear that a flat slab, of 20 ft. span and a thickness of 8 in., contains per panel 784 lb. of reinforcing steel, if designed according to Grashof for a total dead and live load of 300

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lb. per sq. ft. The writer has shown that Grashof's moments for the center and the sides of the square are $\frac{W.L}{48}$ and $\frac{W.L}{24}$, $W = 400 \times 300 = 120\,000$ lb., $L = 20$ ft., and $\frac{W.L}{48} = 50\,000$ ft-lb. and $\frac{W.L}{24} = 100\,000$ ft-lb. for the entire width of the panel, or per lin. ft., 2 500 ft-lb. and 5 000 ft-lb., respectively.

Adopting an allowable fiber stress for the reinforcing steel of 13 000 lb. (although it is too high for mild steel where a factor of safety of 4 is desired), and assuming the distance of center of reinforcing to the compression face as 7 in. (which is a trifle too small for the center of the slab and considerably too large for the sides of the square), we find that the sectional area of the reinforcement for a bending moment of 2 500 ft-lb. per lin. ft. in the center of the slab is 0.33 sq. in. and double the area is required at the sides of the square.

Where the negative reinforcement is obtained by bending all bars up at the quarter point, and continuing the rods to the quarter point in the adjacent panels, the weight of the steel reinforcement per linear foot of the panel in one direction is $0.33 \times 3.4 \times (20 + 2 \times 5) = 34$ lb., and for the entire panel in both directinos $34 \times 20 \times 2 = 1\,360$ lb. It is very poor practice to bend up all bars; this would not be permissible in any girder construction on account of reversal of moments which may occur by settlements and unfavorable loadings in other panels. Assuming that one-third of the bars are straight and extend only 1 ft. beyond the center line at each end, and that, therefore, extra rods are required on top over the sides of the square, we find that at least 340 lb. of additional reinforcement must be placed in each panel, or a total of 1 700 lb.

Hence, according to Grashof, it requires in continuous panels at least three times as much steel as used in the Turner system, provided that the statement in Table 15 in regard to the Turner system is correct. Inasmuch as we have to design for the case of only two panels loaded side by side, it requires a great deal more reinforcing than the 1 700 lb., not counting the additional reinforcing which is absolutely necessary in the columns to overcome the bending stresses produced by one-sided loadings.

The writer begs not to be misunderstood as being an opponent of flat slab constructions. He thinks he was the first to use flat slabs on a large scale, for he built two slabs, 100 ft. square, at the new plant of the Proctor and Gamble Company's soap works at Armourdale, Kans., in 1903, and has built many others since. He protests, however, when engineers and contractors (who, as a rule, have all to gain and nothing to lose), represent by versatile agents that flat floor constructions, of designs as advocated by Dr. Eddy, are as good as, or

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better than, girder constructions, which actually have a factor of safety of 4 or more, if they are designed according to the Chicago Building ordinance, for example, and if they are constructed right. No wonder that they can show to unsuspecting architects and owners a great saving over all other designs, and, being able to mention a great number of examples of buildings which did not fall down (having a factor of safety of about 2), are believed to be by the owners and architects very wizards in the art of reinforced concrete construction. It is true, however, that flat slabs show a slight saving (even if they are designed for a factor of safety of 4) over girder construction of the same factor of safety. The writer has not the reputation of wasting any material in the structures which he designs and builds; being a contractor, it would simply diminish his profits, yet he can mention a number of examples which were actually loaded with about four times the total dead and live load without breaking down.

In December, 1902, a girder in the Salvation Army Building, in Cleveland, of 24 ft. span and designed for a load of 100 lb. per sq. ft., was tested with 100 000 lb., which is about $3\frac{1}{2}$ times the total dead and live load. The girder showed some cracks, but no indication of a near failure, and the permanent deflection was $\frac{1}{4}$ in.

In the spring of 1905 the writer built a furniture storage house for the Brown Transfer and Storage Company, at St. Joseph, Mo. The structure was designed for a live load of 100 lb. per sq. ft. In the fall of 1905, when the writer started another building for the same company, he found that the building was rented to the Parker Grocery Company, which lost its own building by fire, and was loaded all over to the ceiling with canned goods weighing at least 600 lb. per sq. ft., so that both floors and columns withstood nearly four times the total dead and live loads.

In 1905 the writer designed and constructed a building for the Seng Company, Dayton Street, Chicago. The building was designed, according to the new Chicago ordinance for reinforced concrete, for a live load of 100 lb. per sq. ft. On account of some failures of concrete buildings all over the country at that time the owner refused payment on the building as the work progressed until such time as the writer could prove that the building had a factor of safety of 4. In the presence of a great number of engineers and architects, the writer tested one panel, only one month old, in December, 1905, with 600 lb. per sq. ft. The girders and slabs deflected and cracked considerably, but did not fail under the load, which remained there 24 hours. The permanent deflection was only $\frac{1}{8}$ in.

These examples should suffice to show that girder and slab constructions, correctly designed and built, have a factor of safety of at least 4, and there is no question that flat slab constructions can be made as safe as any others, but the writer claims that flat slabs designed according to Mr. Eddy's formula have only a factor of safety of 2, under the most favorable circumstances.

Mr. Hatt. W. K. HATT, M. AM. SOC. C. E. (by letter).—The recent extension of testing operations to completed structures has shown, what many have known, that reinforced concrete structures carry their working loads in service by the help of tensile stresses in the concrete. The stresses in the steel may not be more than one-half or one-third of those which would be calculated by the ordinary formulas, which neglect these tensile stresses.

For the purpose of standardizing design, and as a matter of safety, the conventions of the building codes properly omit these tensile stresses. They cannot be ignored, however, when the complete action of the structure is to be accounted for under working loads.

The variability of the quality of concrete not only makes it desirable to omit these tensile stresses in calculations for design, but it also renders the steel stresses, which are measured in tests of buildings, a very uncertain factor; that is to say, the measured stresses in the steel depend very largely on the quality of the concrete. Good concrete may relieve the steel of the greater part of its expected moment of resistance.

It follows, therefore, that, in judging the mechanical action of any type of structure, more attention should be paid to the measured compressive stresses in the concrete than to the measured stresses in the steel.

Mr. Eddy. H. T. EDDY, ESQ. (by letter).—Mr. Godfrey has taken the opportunity to repeat his objections and warnings against the use of flat slabs, and especially to insist that, whatever may be the true theory of an interior panel of a flat slab of indefinite extent and uniformly loaded, that theory cannot apply either to a single loaded panel or to an exterior panel, and, further, that the theory under discussion involves reliance on direct tension stresses in the concrete.

The full treatment of these questions involves a multitude of details, including bending moments in columns, cantilever action, direct and indirect stresses in concrete, bond shear, shock due to moving loads, the conditions under which beam action prevails over slab action, etc., etc. These questions and criticisms have been raised before by Mr. Godfrey, in connection with the discussion of Mr. Nichols' paper,* and have been answered in detail. Ultimately, dependence must be placed on the facts as they appear in test and practice, rather than on *ex cathedra* statements of what must occur as a consequence of views held on theoretical questions.

The crux of the entire matter seems to lie in the question of direct and indirect stresses in concrete, the latter being those called into play by bond shear and the former by bending.

* "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," *Proceedings*, Am. Soc. C. E., for April, 1913.
(This paper has not yet been published in *Transactions*, owing to the failure of the author to close the discussion.)

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The indirect tensile and compressive stresses in the concrete due to bond shear enable the rods of the belts which cross each other to co-act with each other under tension and cause the slab to exhibit effects similar to those measured in flat plates by Poisson's ratio. It is evident that as long as the bond is intact at the surfaces of the rods, the indirect stresses—which necessarily have the same numerical intensity as the bond shear—do not exceed the strength of the concrete, and the mechanism for the same kind of interaction exists in the slab as in the plate. This phenomenon, therefore, must be taken account of in one case as much as in the other. Consequently, the value of K for slabs is inextricably bound up with the question of bond shear and indirect stresses in the concrete in the tensile zone. No general discussion of the numerical value of K will be taken up at this time, other than to remark in passing that, in order that a homogeneous solid may remain of constant volume, it is necessary from geometrical considerations to have $K = 0.5$, a value which undergoes greater or less diminution in case of various actual solids, according to their volumetric elasticity.

Likewise, in order that a material sheet or surface may remain of constant area, it is geometrically necessary that $K = 1$, where K refers to lateral distortion in the plane of the sheet or surface, as it does in a slab, and has nothing to do with changes of thickness of the sheet. For sheets of various materials, this value of K will be diminished according to its areal elasticity in resisting forces tending to stretch it in its plane.

All reinforced concrete construction is based on the effectiveness and reliability of bond shear. Without its action, no beam or slab could endure, but its action in a slab, not only effects all the results it produces in a beam, but must necessarily cause the slab to exhibit additional properties due to the interaction of multiple-way reinforcement consisting of wide belts of rods in contact with each other. In case the belts are made of numerous parallel rods with a spacing comparable to the thickness of the slab, the slab is sufficiently fine-grained to act very much as if it were of uniform texture but formed of some material not the same as either of its constituents.

It is entirely beside the mark for Mr. Godfrey to argue that, because this composite material has in it reinforcing rods, and Poisson's ratio does not apply to rods considered separately, that this ratio, therefore, has no application to a material like a slab of which rods form an integral part by their intimate union with the concrete, just as truly as if the reinforcement were made into a single sheet and combined with the concrete, although, of course, the value of K would not be identical in the two cases.

What the physical and mechanical properties of such a slab may be cannot well be predicted with certainty, although it may be pos-

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sible to give a rational explanation of what experiment may show its new and unforeseen properties to be. Whenever such a slab undergoes flexure which causes it to assume a dish-shaped deformation, the peculiar and unexpected properties appear; but it is just on this question of dish-shaped deformation in a slab that Mr. Godfrey has expressed views least in accordance with the facts.

Suppose, for example, a long building with two rows of columns parallel to the long sides, and panels nearly square, so that there are three tiers of panels lengthwise of the building, one long interior tier being between the two long wall tiers. Now, suppose heavy loads to be placed on three panels of a tier or bay extending across from wall to wall. If the writer understands Mr. Godfrey, he thinks that such an arrangement of loading would produce a critical stress in a mushroom slab much greater than would occur if the loading were removed from either two of the three panels. It might do this were it not for the wall supports and the stiff heads; but in fact the reverse is true. The walls entirely prevent the formation of the ends of any such cylindrical trough across the slab, and the stiff heads and walls will tend to accentuate the hollows parallel to the long walls, rather than those perpendicular to them.

The experimental test on which Mr. Godfrey relies to prove his contention has little or no bearing on the slab question, for similar reasons.

The panes of glass on which he experimented* had no continuous support at the edges, such as a slab has, to say nothing of additional local stiffness at the points of support furnishing at these points moments of resistance several times as great as those found elsewhere. The plates were not clamped to practically immovable supports so as to make them act in a manner at all similar to a floor slab such as that under discussion. Consequently, these plates would not act like a floor slab at all, and the assumption that they would is entirely misleading. As just stated, the behavior of glass plates supported in this manner would be the reverse of the slab, in that the troughs in the one would be accentuated in a direction different from those in the other.

This and other matters in the discussions connected with Mr. Nichols' paper directly controvert the statements of Mr. Godfrey, which he evidently regards as incontrovertible, as he has made these statements a number of times and they have never been controverted, as he says; and all this will occur without contravening any law of Nature.

All engaged in concrete design and construction should be equally desirous, with Mr. Godfrey, that it should be conservative and safe, and also that it be adopted where it can safely be done at a cost so

* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1361.

moderate as to permit fire-proof in place of combustible construction, as it cannot be if there are unreasonable requirements. Mr. Eddy.

It is not lightly to be assumed that any concern which has been responsible for the design of about two thousand structures of this character without failure, and with inside opportunities for knowing with certainty what can be depended on, and having also enormous financial interests at stake, may not be a good judge of what can and ought to be done in this field.

Mr. Godfrey refers to a paper by the writer entitled "A Comparative Test of Two Full-sized Reinforced-Concrete Flat Slab Panels,"* and tries to convey the impression that the manner of loading and the design of the columns were responsible for the good showing made by the mushroom slab. He states that the test slab was loaded with balancing loads, meaning by that that the load on the over-hang beyond column centers reduced the stress in the central panels so greatly as to make the stresses in the steel small and misleading.

It will be noticed that Mr. Eckles asserts that his experiments showed greatly increased central deflections due to extending the loading into the panels at the sides of the panel tested. These contradictory views cannot both be true.

The truth is that neither of them represents the facts. If there is anything that has been established by numberless load tests of mushroom slabs, it is that in the standard mushroom slab the central deflection and stresses of an interior panel are not affected to any appreciable extent by the loads on surrounding panels. A very small effect has been thought to be discoverable at the centers of panels situated diagonally, but no mutual effect between panels beside each other.

This fact, which would at first sight seem inexplicable, is apparently the result of a close balancing of two opposing effects, *viz.*, any load just outside a panel will have a tendency to lift the center of the panel, but, at the same time, it will tend to increase the deflection of the side belt between it and the center of the panel. Any such deflection at the side belt will have a tendency to increase the deflection at the center of the panel. Whether the lifting or the increase of deflection preponderates depends on the relative rigidities of the belts and the slab and column heads, with the result stated above. Not having at disposal sufficient data in other types of construction to state conclusions with certainty, the writer has refrained from doing so.

The panel tested by Mr. Eckles was at a corner of a slab. It may have been a mushroom slab in general design without having any features of the system in this corner panel beyond one mushroom head at one of the four corners. More details are needed in order to show that the system is in any way responsible for the results of Mr. Eckles'

* *Engineering News*, March 27th, 1913.

Mr. Eddy.

test. The writer has not attempted to give an analysis for such a panel. It is customary to design wall panels differently from interior panels, but that is no argument against the approximate correctness of the analysis for interior panels.

To return from this digression to Mr. Godfrey's criticism that the results of the test slab were due to balancing loads on the over-hang: The tables and diagrams of the test show that in Loads 3, 4, 5, and 6—which were the ones particularly relied on for showing the behavior of the slab—the loading resting directly on the panel was more than twice as great as that on the entire surrounding over-hang, and, in the case of all the other loads, that on the panel itself was greatly in excess of that on the over-hang. These, consequently, were not balanced loads, and besides, there was no loading whatever on the areas above the column heads for Loads 1 to 6, inclusive, and no large loads later. The loads were applied over an area in the form of a Greek cross, with the intention that whatever load might be applied would be effective in causing bending moments and be situated so as to be less favorable to the mushroom slab than to its competitor.

The columns were 18 in. square and 5 ft. 6 in. from top of footing to bottom of slab, and not, as Mr. Godfrey asserts, "about half as thick as their height." Their diameters, as well as those of the mushroom heads, were in the usual relation to the span. This structure corresponded to a basement floor slab, but, because there were no upper floors, the columns necessarily had very small load stresses per square inch. That, however, did not affect the resistance the columns would afford to unbalanced moments, except to render the columns subject to actual tensile stresses in their reinforcing rods by reason of the small superimposed load. The columns were at a disadvantage for this reason and also because the unbalanced bending moment in the slab was not resisted by the stiffness of columns extending upward as well as downward from the slab, as would occur in a building. That may possibly be regarded as partly offset by the fact that the columns were only 5 ft. 6 in. long, but there was no such divergence between usual conditions and those of this test as Mr. Godfrey asserts.

Nevertheless, this test was not made (as seems to be assumed) to demonstrate the strength of this form of construction, *per se*, but to make a comparative test between it and another form of construction, and to demonstrate that though one operated on the principle of the beam the other did not.

The writer had nothing whatever to do with the design of the slab or the arrangement or carrying out of the test, except to suggest the Greek cross as the ground plan of the loading. The data obtained were submitted to him for discussion. Mr. Godfrey or any one else has the opportunity to discuss them and come to any conclusion the figures will support. Innumerable load tests of standard mushroom

slabs have been made. This test was not made to find out anything about the elastic behavior of mushroom slabs, about which there was any real doubt or lack of knowledge by those designing these structures, but to put the facts in evidence, and further to establish their manner of failure when stresses are carried beyond the yield point. This last was a most important piece of evidence which had not heretofore received such experimental certification as to settle it beyond dispute. Mr. Eddy.

It would seem on the face of it as if this test on a full-sized slab, in which all the details are carefully tabulated by reliable observers in such form as to admit of complete comparison with any other tests, might at least afford as good a foundation on which to base conclusions as the experiments made by Mr. Godfrey on a couple of glass plates under conditions not in accordance with those obtaining in floor slabs, and yet Mr. Godfrey intimates that absolute dependence is to be placed on the indications of his glass plates, and that the slab test should be disregarded as untrustworthy.

It may be that Mr. Godfrey believes that this test "has not even a theoretic interest", but he may find it difficult to persuade any to agree with him except those who are so wholly blinded by prejudice or interest as in fact to be unwilling that any reinforced concrete structures shall be built for use unless designed in such an innocuous manner as to be "out of the running."

Mr. Godfrey has also repeated his unsupported statements as to the unreliability of reinforced concrete construction under rolling or jarring loads, and seems to regard his mere assertion as sufficient to settle the question in this case also. In order to show how groundless is this opinion, which apparently has no other foundation than his own say so, the writer would quote from W. K. Hatt,* M. Am. Soc. C. E., as follows:

"It is well known that concrete, because of its lack of elasticity, absorbs or deadens vibrations, and the sound caused thereby. It is not probable that vibrations reach the steel. The speaker has knowledge of many experimental attempts to loosen the bond by shocks and vibrations. So far smooth bars encased in concrete that have been subjected to shocks and long-continued vibrations seem not to have lost any of their original strength of bond. Likewise the concrete on the compression side of a reinforced concrete beam that has been loaded and released from load some 2 500 times to high working stresses seems not to have been substantially weakened thereby."

For, far as these experiments go, they give no ground whatever for Mr. Godfrey's statement, into which he would seem to have been led by applying a line of reasoning derived from the behavior of all-steel structures which is inapplicable to reinforced concrete. An impact

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. III, 1907, p. 60.

Mr. Eddy.

or blow at any point of a steel bridge is propagated longitudinally along elastic members extending linearly from the point, and it goes practically undiminished to the farther ends of those members, where it is subdivided among other members and propagated still farther. An allowance of 80 or 90% is usually added for impact to the static effect of a moving load.

Impact, or the effect of a blow at any point of a reinforced concrete slab, however, is entirely different from this. In the first place, the effect of the blow does not travel in one direction only, but in all directions radially from its point of application, so that in a very thin slab, its effect at any other point would be inversely as the distance, and in a very thick slab inversely as the square of the distance. This would make the allowance for impact in a slab very small.

Secondly, the effect of impact must be inversely proportional to the weight of the body receiving the blow. It may be assumed that steel costs from twenty to thirty times as much per ton as reinforced concrete, and the effect of impact on a steel structure costing the same as a slab would be from twenty to thirty times as much, for that reason.

Thirdly, the continuity and stiffness of the slab greatly reduce its vertical, lateral, and torsional deformations below those of a steel structure. The work done during impact, and its effect, depends on the amplitudes of the deformations. In particular, the horizontal resistance of a slab is many thousand times that of a steel structure. The vibratory energy absorbed by a slab during an impact is consequently small.

Fourthly, the small amount of energy which is absorbed is not transmitted (as it is in a highly elastic and resilient structure) to a considerable distance in the slab, but, owing to the nature of concrete, is dissipated near its source, transformed into heat, and rapidly absorbed.

Fifthly, slabs are tough, and not brittle, like terra cotta, for example, so that, in cases where great weights have fallen on them, little effect has been produced, whereas brittle slabs have been smashed under such circumstances, and have failed.

The concrete of the compression zone is such a shock-absorber as to protect the tension zone from jarring and vibration, both as regards steel in tension, and concrete as well.

For all these reasons the shock which a rolling load imparts to a slab is inconsiderable, and is absorbed and dissipated so readily that it is a negligible factor, instead of being such an important and overshadowing consideration (as Mr. Godfrey says) as to prevent its safe use in railroad viaducts and the like.

In support of these contentions the facts respecting the concrete railroad bridge, Fig. 21, which carries the track of the Soo Line between

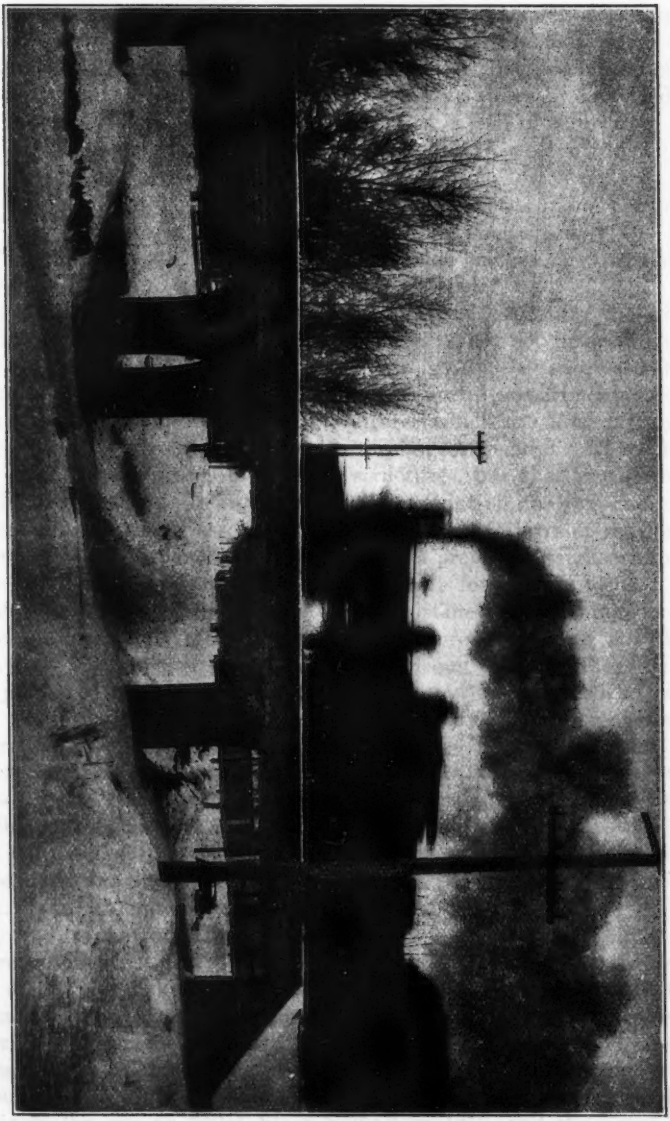

FIG. 21.—SOO LINE RAILROAD BRIDGE OVER WILSON STREET, AMHERST, WIS.

Chicago and Minneapolis over Wilson Street at Amherst, Wis., are instructive. It is a slab or girder, 18 ft. 6 in. wide and 2 ft. 3 in. thick, with reinforcement in excess of 1%, acting largely as a girder, but with some slab action, having a central span of 25 ft. 9 in. and designed to carry the heaviest locomotives with inappreciable deflections. It would be extremely difficult to cause failure by any loading that could be placed on it. It would require a load of more than 600 tons on this panel to cause a unit stress in the steel of 16 000 lb. This bridge has given such excellent satisfaction for years that the Superintendent of Bridges and Buildings of the Soo Line considers reinforced concrete as the desirable material for such spans. Mr. Eddy.

Particulars respecting other satisfactory slab bridges carrying street-railway traffic are to be found in the writer's book.*

It would seem that Mr. Godfrey's opinions respecting the availability of reinforced concrete for such purposes are not in accord with successful practice, and are not such as should be expressed with vehemence until they are supported by careful experimental tests, rather than by vague references to failures which are used merely to cast discredit on reinforced concrete design in general. There have been lamentable failures of all-steel structures, but that is no reason for discarding them. It is reason for discriminating analysis and careful design. The same lesson is to be derived from failures of reinforced concrete design, rather than to use them as a pretext for indiscriminate condemnation of reinforced concrete as unsafe for moving loads.

Mr. Eckles must have made some mistake in ascribing to the writer any previous attempts to treat reinforced concrete flat slabs. It is true that a paper on "Thin Homogeneous Circular Flat Plates" was published by the writer some years ago, but it contained no reference to any application of the flat slab theory. In the paucity of theoretical developments which could possibly furnish the basis for slab theory, this paper was used again and again in current engineering literature for that purpose, but the writer was in no way responsible for this, and in fact found so much in these applications with which he was at variance, for one reason or another, that he was induced to investigate the theory of reinforced flat slabs, as he has done in his book. The attempt now under consideration is his first effort in that line, and is entirely independent of his previous paper, and any reference to "his past efforts" simply reveals the lack of information of the one making such reference, and has no foundation in fact. When Mr. Eckles mentions discrepancies between the writer's former theories, and conclusions based on facts, it would be well to cite references, as the writer cannot recall any work of his to which these remarks apply. The only development of slab theory in any work of his is in the one on "Flat Floor Slabs," in 1913, and the paper now

* "The Theory of Flat Slabs," pp. 67-69.

Mr. Eddy.

under discussion is the application of the theory there developed to test results in several slabs. There is certainly no discrepancy here, such as Mr. Eckles would have us believe.

Mr. Thompson apparently does not understand the object the writer had in view in his paper. It was to show that test results as to deflections and stresses, as far as they were available, were in such accord with the analysis which he had proposed in his book that a reasonably close practical agreement existed between them, especially in the case of the mushroom system with which he was in a position to obtain inside information more complete than was the case with other systems. At the same time he utilized for comparison such information respecting other systems as was at his disposal.

Mr. Thompson seems to think that if he could show that the test slab of the Northwestern Glass Company's Building was insufficiently reinforced, or over-strained by the test load, or otherwise incorrectly designed, he would thereby discredit the computations in the paper; but such is not the case.

It is no doubt the truth, as Mr. Thompson alleges, that it is possible to draw further conclusions from the test data given than were drawn by the writer, and it may be surmised that the four conclusions stated by Mr. Thompson at the beginning of his discussion do not by any means exhaust the list. It is assumed that wall panels must necessarily differ in design from interior panels, and that every one conversant with slab design is perfectly aware of this, and hardly needs to have it called to his attention. It is believed that wall panels invariably differ in this way. The amount of that difference, however, is dependent to a large extent on the character of their connection with the walls and pilasters.

The deflections of Panel A shown in Fig. 7, when compared with those in Panel D, do not indicate that in this case the moment of resistance of Panel A as actually designed needed to be increased by more than a slight percentage to make its resistance equal to that of Panel D. As this is a question not yet entirely amenable to analysis, it remains to a very considerable extent to be determined in each case by the judgment of the designer. Other types of slabs having more flexible connection with the columns would no doubt require greater additional stiffness in wall panels than was needed in this case.

The principal questions at issue, however, as seen by Mr. Thompson, appear to be based on his two assertions that Equations (*a*) and (*b*) do not in fact give results that accord with tests, and as the principal support of that assertion, the further assertion that the observed stresses at gauge line 111 were not abnormal, but such as to be expected. The writer will proceed to meet these assertions at some length and then treat other matters broached by Mr. Thompson.

Mr. Eddy.

It should be stated first that the writer had nothing whatever to do with the design or testing of this slab, nor with that of any of the other slabs treated in this paper, and it is a subject of regret with him that the number of gauge lines over the column heads was so small, and that they were so irregularly distributed. However, he was compelled to make the best use possible of the data at his disposal, and he is still of the opinion that his interpretation of the results is substantially correct, as he will proceed to show by making use of analogous data from an unpublished test which has been put at his disposal since the paper was written.

Additional Data.—This is the case of a slab, 7 in. thick, with panels 18 ft. square, design load 150 lb. per sq. ft., and loaded at once over four panels forming a square, with a load which if uniformly distributed would have an intensity of 151 lb. per sq. ft., but disposed so as to leave several open spaces. It is estimated that the stresses in the steel over the center column were such as would be produced by a uniform load somewhat in excess of the design load. The four loaded panels were two wall panels and two adjacent interior panels, just as in the Glass Company test.

Fig. 22 is a plan of the central column head and the slab rods crossing it, with the outer ring rod 8 ft. in diameter, the inner ring rod 4 ft. 6 in. in diameter, and the cap 3 ft. 4 in. in diameter. The plan also shows the relative positions of the numerous 8-in. gauge lines, most of which were in the semi-circle most distant from the wall, and the length of the laps drawn to scale. Each belt consisted of thirteen ¾-in. round rods, except the direct belts which cross the wall panels from pilaster to column, and these had sixteen rods each.

The measured stresses, with the mean embedment of the axis of each rod at the extremities of the gauge lines, are given in Table 16.

Gauge lines 14 and 44, were on the top and bottom of the same rod, as were 15 and 42 also, from which it is evident that the other observed stresses on the tops of rods are most certainly in excess of the mean stresses to which the rods were subjected. Especially must this be true at the edge of the cap, where there is a sudden change in the flexibility of the slab. For that reason it is quite uncertain what actual mean stresses are indicated by the stresses which were observed on the upper surface of the rods crossing the edge of the cap radially; but we may be sure that they are much smaller than those corresponding to observed elongations.

It thus appears that some part at least of the excessive stresses apparently found here are due to placing the gauge lines on the upper surface of the rods, and the explanation which has been offered for these excessive stresses as being due to the thick boss formed by the head is needed as a reason to account for only a fraction of the observed results.

Mr. Eddy.

Equations (a) and (b), proposed for computing the maximum stresses in the slab rods of side and diagonal belts, respectively, in the head, give results which do not differ greatly from each other.

Let $W = 50\,000$ lb., $L = 216$ in.
$A = 13 \times 0.11 + = 1.4365$ sq. in.
$d_s = 5.5625$ in.

Then Equation (a) becomes

$$f_s = \frac{50\,000 \times 2 \times 216}{800 \times 5.56 \times 1.43}\left[\frac{3\,(216)^2}{(216 - 40)^2} - 1\right] = 12\,000 \text{ lb. per sq. in.};$$

but $L'^2 = 2\,(216)^2 - (96)^2 = 102\,528$, and $B' = 265.4$, and Equation (b) becomes

$$f_s = \frac{50\,000 \times 216 \times 3.37}{400 \times 5.56 \times 1.43} = 11\,500 \text{ lb. per sq. in.}$$

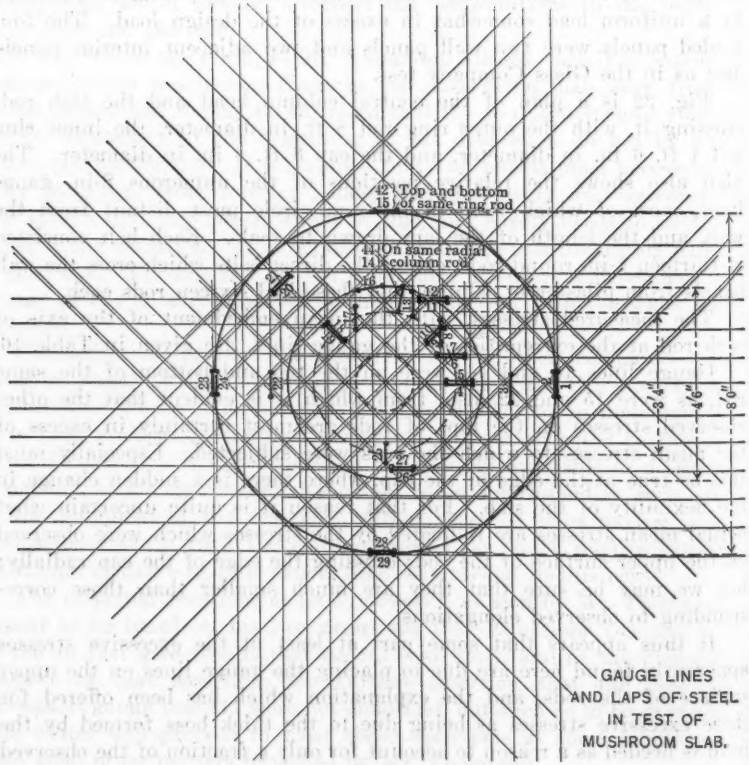

FIG. 22.

TABLE 16.

Gauge line No.	Embedment, in sixteenths of an inch.	Observed unit stress, in pounds.
Center Rods of Belts.		
4	23	8 650
5	23	8 150
6	23	9 200
7	23	7 850
9	11	9 150
10	11	9 100
18	18	8 850
19	18	9 300
25	30	8 100
Other Slab Rods.		
1	56	1 800
2	56	1 700
3	44	3 500
8	27	5 000
11	22	6 850
12	23	5 600
13	28	6 150
17	31	6 250
20	33	2 000
21	33	8 300
22	57	2 650
23	57	1 450
24	57	1 000
26	27	5 700
27	27	5 150
28	58	800
29	57	1 300
Radial and Ring Rods.		
14	48	3 350
44	.....	2 300
15	57	1 850
42	.....	550
16	33	3 750

The reason that these computed values are in excess of those observed, lies in the quantity of steel to be found in the laps, as shown in the plan, which quantity is evidently largely in excess of that for which the formula was computed, as will be seen on referring to page 32 of "The Theory of Flat Slabs," where it appears that the mean cross-section of the slab steel was assumed to be 4.25 times the area of a single belt. The case in hand certainly exceeds this, as all the belts are lapped as far as beyond the inner ring rod, with a consequent reduction of the stress at the edge of the cap of perhaps 25% below the value computed by Equations (*a*) or (*b*).

It should be noted that the observed results show some diminution of stress with the embedment, but, in the slab rods themselves, this is

Mr. Eddy.

by no means proportionate at the edge of the cap to their approach to the neutral axis, although it is more closely proportionate elsewhere.

All these gauge lines were located around the central column corresponding to Column 45 of the Northwestern Glass Company Building. Besides these gauge lines, seventeen more were observed at three other column heads corresponding to Columns 36, 37, and 44 of Panel D, on the edge of the loaded area. The gauge lines, with one or two exceptions, were located (as were those around the central column) so that the edge of the cap nearly bisected each of them. The largest pair of stresses observed showed 7 400 and 7 700 lb. per sq. in., respectively, being on parallel adjacent diagonal laps. The next smaller observed stresses were 6 450 and 6 250 lb. per sq. in., respectively, with no other values as large as 6 000. In general, the values of the maximum stress in these gauge lines varied from 70 to 80% of those over the head of the central column. The two first mentioned appear to be abnormally large for some unknown reason as others similarly situated were much smaller.

Again, in Mr. Lord's report of the test of the Deere and Webber Building,* Fig. 14 shows cracks on the upper surface of the slab within an inch or so of the edge of the caps in the loaded area; but these cracks cannot be said to lie more outside than inside of the edges of the caps. It is clear, therefore, that those radial gauge lines that cross the edge of the cap in Fig. 22 will unquestionably show maximum steel stresses. Gauge lines 13, 14, and 44 give no indications that maximum stresses are to be found outside the cap. These additional data are thus seen to be confirmatory of the substantial accuracy and applicability of Equations (*a*) and (*b*).

Recalculation of Equations (a) and (b), in the Northwestern Glass Company Building.—Consider now the effect of the distribution of the actual loading. Suppose that Load 3, which consisted of 380 000 lb., were at first uniformly distributed over the four panels, each 16 by 17 ft. Next, let the loading on an area 7 by 8 ft. over the central column, and amounting to about 20 000 lb., be entirely removed and be piled on the surrounding parts of the slab. This will practically leave the cantilever portion of the slab around the column without loading, and will place this loading outside the inflection lines which surround this column. If similar removals occur at other columns, so that the loads are nearly symmetrically disposed on the central parts of the panels and the side belts, then it will be of little consequence, as far as the stresses in the cantilever portions of the slab are concerned, whether the loads outside the lines of inflection are uniformly distributed or have wide open ways across them; the stresses in the cantilevers will necessarily be increased by removal of the loading from the cantilevers to the other parts of the slab.

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, 1911.

Mr. Eddy.

Mr. Thompson says that the stresses "are more affected by the [total] amount of load in the panel than by their position". That certainly is not the fact, for, to take an extreme case, the loads might possibly be applied directly to the tops of the columns and produce no stress in the slab. The fact is that removal across the lines of inflection affects the stresses in the cantilevers, and removals within other portions of the panels affect the moments in those portions of the panels where the removals occur, but as long as they are symmetrical, such removals have little or no effect on the cantilevers.

In the paper it is in effect assumed that this removal added more than 10% to the effect of the load, as far as the stresses in the steel at the edge of the cap of the center column are concerned, and such allowance is believed to be not too great. Mr. Thompson disagrees with this on general reasoning, based apparently on principles applying to beams simply resting on supports, instead of firmly fixed to stiff supports, in which the position of the lines of inflection would be affected by the distribution of the load to an extent which is not the case in slabs, where the lines of inflection are practically fixed in position and remain the same for different distributions of load. Mr. Thompson, however, apparently states the truth inadvertently when he asserts that "the same load uniformly distributed would not have caused larger stresses than was caused by the load placed as shown in Fig. 5", for, as we have just seen, it would certainly have produced smaller stresses.

In case the lines of inflection are practically unaffected by shifting the load from the cantilever, as just described, the stresses at the edge of the cap will be increased, and it will also be justifiable to neglect the effect of any change in distribution of loads due to the center aisles, 18 in. wide, when treating the cantilevers, though perhaps not in treating other portions of the slab.

On repeating the calculation of the maximum stress at the edge of the cap, as given in the paper, the figures were found to be somewhat inaccurate by reason of using an incorrect value for the clear span between the caps. The correct figures are:

Diameter of cap $= 50$ in.; diameter of head $= 87$ in.

$L_1 = 204$ in., $L_2 = 192$ in., $B_1 = 204 - 50 = 154$ in.

$L'^2 = (204)^2 + (192)^2 + (50)^2 = 86\,049.$

$L^2 = (204)^2 + (192)^2 = (280)^2$. $B' = 280 - 50 = 230$ in.

If $W = 106\,250$, $d_3 = 6.5$ in., and $A_1 = 15 \times 11 + = 1.6567$ sq. in.

Then, by Equation (a), $f_s = 20\,700$;

and by Equation (b), $f_s = 19\,500$;

which values may need to be increased somewhat for lack of laps, as previously stated.

Therefore, a mean value of 20 000, say, must replace the erroneous value, 17 000, originally given.

Mr. Eddy.

It would be interesting to see the calculation by which Mr. Thompson says a value of $f_s = 15\,400$ is obtained by the application of Equation (*b*). The only change which he has suggested in the computation consists in scaling down the load of 106 250 lb. per panel, which he says is too large, though that does not appear to be the fact, as has already been shown. However, proceeding on that basis for the moment, let the load be scaled down to 96 000 lb. per panel, then, provided the originally computed value given in the paper ($f_s = 17\,000$ lb.) were correct, we should find $f_s = 15\,400$ lb. Recalculation, however, makes the value of f_s at least 19 500 instead of 17 000 lb., and the scaled value would consequently be 17 600 instead of 15 400, a discrepancy of 2 200 lb. per sq. in., or more than 14 per cent. It must be that Mr. Thompson did not actually compute this by the formula, but accepted without investigation the erroneous figure in the text.

There were, in fact, no laps whatever in the slab steel over Column 45, and the general formula, as discussed on page 32 of "The Theory of Flat Slabs," would need to be modified slightly for that reason, so that, in applying Equations (*a*) and (*b*) to this case, the reduction of the cross-section of the steel in the belts below the mean value assumed in establishing the formula would require a corresponding increase of the maximum stress at the utmost of, possibly, 20% above that computed from the tentative coefficients in Equations (*a*) and (*b*). Indeed, 10% in addition to the computed stress is sufficient to cover the abnormal unit stress of 22 000 lb. at gauge line 111.

It may truthfully be said, therefore, that there is good agreement of Equations (*a*) and (*b*) with stresses in this test, even if the actual stresses be assumed to be considerably in excess of any observed.

Abnormal Stresses at Gauge Line 111.—Mr. Thompson has misunderstood the facts stated in the paper respecting the stress at gauge line 111, as he has confused the elbow rods with the slab rods. The eight radial elbow rods, shown in Fig. 15 under the slab rods, are of 1⅛-in. round steel, and are of great stiffness compared with the slab rods. As stiffness is proportional to the fourth power of the diameter, one of these rods is more than 80 times as stiff as a slab rod. These are the rods that were forcibly pried and held in position in the slab until relieved by the application of the test load, when they acted with that load in inducing stresses in the slab rods which they were unable to produce while acting alone, being held by the shrinkage stresses of the concrete. Thus, after the slab had once suffered flexure by the application of a considerable load, and had had a chance to recover, the radial rods and slab rods would come back to a position of equilibrium and mutual adjustment different from the one they occupied before the application of the load. A renewed application of loading would then show elongations and stresses measured from this new position

of equilibrium corresponding to the loading. If concrete was a perfectly elastic material, however, such an action would not occur. If any such steel stresses were around the edge of the cap at Column 45, as Mr. Thompson states, they could not fail to show at such a gauge line as 107, which crosses the edge of the cap. The assertion that the stress at 111 was not abnormal because it increased and decreased with the load, is inconclusive, because that depends on how much decrease occurred on the removal of the load as compared with the previous increase. The fact is that the decrease of stress at 111 on the removal of the load was so small as to make it apparent that some such action was occurring as has been described. That this is the correct explanation of the observed abnormal stress at gauge line 111 seems to be established conclusively by the observed unit stress of 11 600 lb. remaining after the removal from Panel C of all but 7 000 lb. of the load of 47 100 lb., and an observed unit stress of 10 300 lb. still remaining a day later. This abnormal stress after the removal of the load, such as is not shown at any other gauge line, makes it clear that the stress at 111 was abnormal before the load was removed, and nullifies any reasoning that leaves that fact out of account. Mr. Eddy.

Location of Gauge Lines.—A misconception is likely to arise respecting the matter of stresses inside the cap and outside of it, though disregard of the fact that absolutely sudden changes of stress cannot occur at any point of a rod, because such changes must be more or less gradual from point to point. Owing to this fact, and to the fact that the edge of the cap is comparatively thin, large stresses are to be found for some small distance inside the geometrical outline of the cap, and are not confined to the area strictly outside of it. Indeed, some designers, in order to take account of this, have assumed the effective diameter of the cap as 2 or 3 in. less than its actual diameter. This question has its bearing on the location of the gauge lines with reference to the geometrical edge of the cap.

Again, the single abnormal result cited by Mr. Thompson, and shown in his gauge lines, *A* and *B*, does not accord at all with what is generally known respecting the distribution of stresses at the edge of the cap, and, until more specific information is available, it should be disregarded because a single measurement of this kind cannot be depended on, even were it not known that it is abnormal. It would be quite understandable that whichever of Mr. Thompson's gauge lines, *A* or *B*, lay across a crack would show the greater stress. As seen in the Deere and Webber test, however, that might as probably occur with *A* as with *B*; or it may merely be due to a so-called kink in the rod being straightened out, or merely the extra tension in the upper surface of the rod.

Mr. Thompson's attack on the validity and general accuracy of Equations (*a*) and (*b*) is thus seen to have a very slight foundation in

Mr. Eddy.

test data, and his estimate that the actual stresses may exceed those computed by these equations by as much as 45% may be disregarded.

Mr. Thompson says, respecting Mr. Turner's test slab, that it was broken by loads placed on the center panel, with comparatively little load on the projections. That is the truth, Mr. Godfrey and his balanced loads, to the contrary, notwithstanding. Mr. Thompson criticizes the test because it was not directed toward elucidating the particular problem in which he is interested. Mr. Turner, however, had in view the solution of a different problem, for which he was willing to expend money. Neither was that problem the one Mr. Godfrey would like to have seen investigated. It will be interesting and valuable for these gentlemen to investigate the problems they have in view, but they can hardly expect others to put aside their own pressing problems because they themselves are not interested in them.

It may be stated again, however, that, so far as can be learned from numerous load tests of mushroom slabs, central deflections of panels are unaffected by loading adjacent panels, which, if true, may permit this test to be used to solve some of the problems proposed by these critics. However, it is perfectly evident, from the report of the test and the photographs of the failure,* that Mr. Thompson is mistaken if he supposes that the steel and concrete just at the edge of the cap were ultimately more severely stressed than at points somewhat farther from the center of the columns, for the test slab gave way at points at least as far distant from the cap as gauge line 103, instead of at the edge of the cap, and this was a case of belts without laps.

Mr. Thompson entirely misses the point of the argument, respecting the comparison of the stresses at Column 36, and those at Column 46, which are similarly situated with respect to Load 3. At 36 the stress lines given are those largely due to the loads in Panel C, and at 46 to loads in Panel D. Whatever changes occurred at 46 by increasing the load on D might be expected to be paralleled, were the load on C increased. What is there "absolutely misleading" about this? From the behavior at 46 when the load on D was doubled, it might perhaps be surmised what would happen at 36 were the load doubled on C, which it was not. What misleading conclusion has been here advanced?

Certainly there is nothing so misleading in this as in Mr. Thompson's statement that the slab bar on which gauge line 103 was taken "was one of the outside ones of the diagonal band". In fact, however, the bar was the third one from the center bar and the outside ones were the seventh from the center bar, all equally spaced, and so this bar was not half way to the outer bar, where it was asserted to be by Mr. Thompson. That, unquestionably, is "absolutely misleading".

* "The Theory of Flat Slabs," p. 92.

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Mr. Thompson criticizes the values obtained for stresses in the steel in the diagonal and rectangular bands because, he says they "are calculated from formulas derived without regard to the size of the column head, and, * * * would apply to a construction with a column head equal to a sharp point and one of any size whatever." He says "this assumption, on the face of it, is erroneous". Whether erroneous or not, one thing is certain, *viz.*, the stresses, if correctly computed on the assumption stated, will not become larger on account of making the head of appreciable size. The effect of the head, if anything, will be to decrease the calculated stresses at mid-span, and the error, if such it be, will be on the side of safety, and on that account is unobjectionable, from a practical standpoint, however much it may detract from the accuracy of the formulas. As the criticism is based on what appears "on the face of it", it might be well for Mr. Thompson to make an estimate, if he can do so, as to the amount of the effect produced at mid-span of these belts by ordinary sized heads, when due consideration is given to the fact of the comparative fixity of the lines of inflection, thereby conferring on the central parts of the panels outside the cantilever a semi-independent character, which justifies the assumption as one approximately correct. The final proof of the applicability of the formulas, however, is to be found in their accordance with test results, as has been shown in the discussion of the tests.

Mr. Thompson criticizes Equations (*a*) and (*b*) because the observed stresses at the edge of belts opposite column centers fall short of those computed from these equations. It must be remembered that these equations were derived on the supposition that the steel was as near the top of the slab as it is over the column caps. As it is in fact at a much lower level at the edges of the belts, less stresses will be observed here. This is part of the reason for the occurrence of large stresses at the edge of the cap; but the theoretical stresses thus computed furnish a basis on which to compute maximum stresses in slab steel over the head.

Mr. Thompson objects to the word "theoretical" in this connection. The word correctly designates a result reached from a consideration of the predominating principles involved. It does not mean, when applied to engineering structures, that all factors have been introduced into the premises. It means such simplified assumptions and premises as will take account of the dominating factors and give as a result a reasonable approximation which is accurate enough for practical purposes. As neither Mr. Thompson nor, so far as is known, any one else has proposed an approximate theory in any practical agreement with experiment, it would seem to be the courteous thing not to assail a proposed theory on the basis of assertions which seem not to be well founded.

Mr. Thompson says that a design, such as that of the Northwestern Glass Company Building, does not have a proper factor of safety, the

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inference being that radical changes of design are required, whereas it appears from the preceding analysis and tests that all that is required to bring down the unit stresses in the steel at the edge of the cap, and reduce them by any desired amount, is merely to place additional short rods across the cap between the slab rods, of a length from three-tenths to four-tenths of the span, and no general alteration of design is required, as is intimated.

Mr. Thompson refers at the same time to stresses in the concrete. It is not the object of the paper to go into that subject further than absolutely necessary, and it will be postponed for future consideration, with the remark in passing that, where an absolutely flat slab is not required, every one is aware that increased thickness of slab in the vicinity of the cap will reduce the concrete stresses.

What the writer is especially concerned about in this discussion is, not the close accuracy of the particular computations of this paper (for they are admittedly based on average data which do not necessarily coincide altogether with the particular examples to which they are applied), but to convince those interested in slab construction of the fact that a method has been found for making a rational analysis of slab design, a method which will ultimately enable the engineer to predict results with somewhat the same certainty as is now attained in ordinary structural work. That may require modifications of constants, etc., in his practical formulas, which apply to the mushroom system. Such modifications will be no greater than every engineer uses in bridge construction where a method is used, which is not expected to result in the same final formulas for stresses in all types of bridges.

This paper proposed to discuss "Steel Stresses in Flat Slabs," and see how closely they could be computed by the analysis published by the writer in his book. Mr. Mensch regards the paper as taking up the general defence of the mushroom system. That is an unwarranted enlargement of the scope of the paper. He seems to think that if he can make it appear that any of his numerous objections to flat slabs are admissible, he has thereby cast discredit on the paper in some way. Let it then be said, by way of preface, that Mr. Mensch has not called attention to a single discrepancy between the computed and the observed results. The only discrepancies mentioned by him are between results given and confirmed both by analysis and experiment with what he believes they should be.

The real question at issue, however, is this: can the writer actually compute steel stresses in any one type of flat slabs correctly? We are led to believe that Mr. Mensch thinks he cannot do this, but instead of meeting the main issue directly he introduces a number of side issues. The side issue with which he begins and ends his remarks is the question of the factor of safety in flat slab construction, and the load tests he has made on his own constructions, showing a factor of safety of at

least 4. He also "claims that flat slabs designed according to Mr. Eddy's formula have only a factor of safety of 2, under the most favorable circumstances". Now, this is not so, because, in the case of the test of the St. Paul Bread Company Building, given in the paper, and designed according to the standards of the mushroom system for a load of 100 lb. per sq. ft., one panel was loaded with 415 lb. per sq. ft., without showing signs of distress or any cracks except a few very fine hair line cracks, to be discerned only with difficulty. That observed result would seem to have more weight than any mere estimates on which Mr. Mensch bases his claims, and would tend strongly to make us put no trust whatever in the claims which he has put forth so confidently. According to him, that slab ought to have failed under half the final test load. The truth of the matter is that the materials of which it is composed are used more economically and will carry more load than if disposed as Mr. Mensch proposes in a slab-girder construction, and though he is willing to concede a slight advantage to flat slab construction, weight for weight, his adoption of a wrong theory prevents him from giving flat slabs the credit to which they are entitled. Mr. Eddy.

Mr. Mensch declares that extensometer measurements of steel stresses afford no correct indication of the strength of a slab, and load tests, evidently, in his estimation, afford no reliable indication of what may be expected beyond the loads actually applied, as he believes a slab is subject to sudden collapse from sudden and unforeseen large stresses in the steel which may be expected to develop with slight increments of load; but, were this the case, this phenomenon should have been met frequently in the course of the numerous tests which have been made. So far from this being the case, the fact is that multiple reinforcement is of such a nature that unusual stresses developed accidentally in any rod of a slab must ultimately be distributed and divided among neighboring rods and thus be safely carried.

Mr. Mensch is mistaken in ascribing the discrepancies between observations at symmetrical points in slabs to sudden changes in stress under comparatively small increments of loading, such as have occurred in beams. A study of the stresses given in connection with the test of the St. Paul Bread Company Building, where these discrepancies were observed, fails entirely to reveal any such phenomenon as this. The discrepancies were in general as pronounced at smaller as at larger loads, with no such sudden increases of stress as alleged. This might well be expected in beams when cracks develop, but a crack such as ordinarily occurs in a slab would not have any such observable effect. It would be necessary for cracks to cross one another in the slab in a way such as they practically never do, to permit even the initiation of effects similar in character to those due to single cracks in a beam.

Almost at the beginning of his discussion Mr. Mensch turns from the discussion of slabs to that of beams, which he makes his principal

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side issue. He says that "one may reasonably expect that the discrepancies will be still greater in flat slab construction", because the percentage of reinforcement is less than in beams. The entire purport and effect of the reasoning and the results reached in the paper go to establish the fact that slab action and beam action are different, theoretically and numerically, and it seems almost useless to attempt to reply to criticisms based on a complete confusion of beam action with slab action. Mr. Mensch says that slab action is "simply a combination of continuous beams acting in at least two directions, which beams are connected with each other and with the columns, and the deflection in any point is common to at least two beams acting at right angles to each other." By that statement he means to say that a theory of computation based on such an assumed network of beams combined together would constitute a sufficiently close approximation in its action to that of a flat slab to afford a practical basis for obtaining the stresses in a flat slab.

The writer does not hesitate to deny that there is any such agreement between the results of such a theory and any set of trustworthy experimental observations that Mr. Mensch can produce. Mr. Mensch refers in particular to Danusso as an exponent of this correct theory, and as one who expresses real slab action in a scientific way. Danusso's papers, to which he refers, have been translated into German and published in book form, so that they are perfectly accessible to any one.* As must be evident, such a network of beams necessarily has supporting girders at the edge of each panel, and the system corresponds to a slab-beam-girder construction or else to a slab-girder construction and not to a flat slab construction. Danusso, consequently, makes certain suppositions respecting the stiffness of these supporting girders, as is perfectly proper to do.

The writer has developed a slab-girder theory which is awaiting more complete experimental confirmation before publication, but any such theory must necessarily differ from flat slab theory and it is absurd on the face of it for any one to think that the results of either theory are applicable to another construction from that contemplated in deriving it. Flat slabs without supporting girders are alone under discussion in this paper, and Mr. Mensch's definition of slab action quoted above does not apply to actual flat slab action.

Mr. Mensch says that "the advocates of many flat slab systems declare that the common beam theory does not apply to flat slabs," and that he "has never seen a similar statement in the works of St. Venant, Winkler, Grashof, Föppl, or Müller-Breslau," and that, "in particular, the case of flat slabs was solved by Winkler and Grashof by that theory". Of course, St. Venant, Winkler, and Grashof made no

* "Kreutzweise Bewehrte Eisenbetonplatten," Danusso-von Bronneck, Berlin, 1913, Wilhelm Ernst und Sohn.

statement about flat slabs, for the very good reason that they had not been introduced in their day. Flat plates, however, is a different question. A flat plate is understood to be homogeneous and of the same moment of resistance throughout; a slab is neither of these, but has increased moment of resistance where required locally. St. Venant, the greatest investigator of the theory of elasticity that ever lived, was much busied with the theory of flat plates thick and thin. A full and critical account of his researches may be found in the encyclopedic "History of the Theory of Elasticity and Strength of Materials," by Todhunter and Pearson, occupying pages 833 to 896 of Vol. I, and pages 1 to 282 of Vol. II. He did not treat flat plates on the beam theory at all, but on the basis of the exact general theories of elasticity, without regard to the approximations involved in the beam theory. He was born in 1797 and died in 1886. Grashof, born in 1826, published the second edition of his great book on "Elasticity and Strength" in 1878. This is the edition to which Mr. Mensch has referred elsewhere and is the one always quoted. It contains his investigation of homogeneous flat plates, but nothing about slabs. Those who have not ready access to this work will find the substance of the more important parts of his plate theory in Lanza's "Mechanics." Mr. Eddy.

It is impossible to explain how a man of Mr. Mensch's careful scholarship could have been led to say that Grashof solved flat plates, not to say flat slabs, by the beam theory. Nothing could be farther from the truth. Grashof is regarded as the great exponent of the slab theory, if not the originator of it, as distinguished from beam theory. It is true that Mr. Mensch in a paper of his* has used Grashof's formulas, and may have supposed that they were established as beam formulas, but such is not the case. Müller-Breslau has not treated flat plates or slabs in any of his published works. It is useless to try to have any beam theory of flat slabs accepted on the authority of any of these writers.

Danusso, in his preface, quotes at first the formula of "Grashof's plate theory", as he calls it, and later the Bach-Föppl formula, which is a convenient modification of Grashof's. It thus appears that Föppl used the plate theory rather than the beam theory that Danusso uses.

It is entirely gratuitous for Mr. Mensch to charge up any defects in the Niagara Building, to which he refers, to others than those responsible for its design, and it may be that the writer will be as ready as any one to condemn its faults if such there be, when he is apprised of the details of the design, of which he is as yet ignorant.

Mr. Mensch criticizes the idea of saying that a set of loads extending across a slab would tend to produce a cylindrical trough or depression under it across the slab, and that another set of loads extending across the slab at right angles to the first set will also tend to produce

* *Proceedings*, Nat. Assoc. of Cement Users, 1911, p. 205.

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a cylindrical hollow or trough across the slab under it, so that in the panel where these troughs cross each other the slab will be "dished" and their mutual effect will be for each trough to diminish the other. He declares that this description is "mysterious" and one that "will not enlighten any one", and yet this is the very idea of a cylindrical trough that Mr. Godfrey uses as the basis of his argument in his experiment. The idea is used by the writer merely as a semi-popular form of description, to assist the imagination in picturing this phenomenon, a phenomenon which he has investigated analytically from an entirely different point of view. It was used in order to put forth a form of explanation which would enable the reader to see how the results which had been reached by his analysis might possibly be pictured; and yet Mr. Mensch says this explanation is used because the writer does not care to express real slab action in a scientific way. It is evident that the critic has not followed the analysis, and is not even aware that that is exactly what has been done in the writer's book, where he has based his analysis on the general equations of equilibrium of the forces, shears, and moments acting on the external surface of an infinitesimal element of the slab, as is necessary in order to make a perfectly exact and rigorously scientific investigation of the slab.

It is evident that, with such divergent views of what constitutes slab action and slab theory, it would be useless for the writer to try to come to any understanding with Mr. Mensch within limits now at his disposal, especially when he insists, as he does, on the theoretical identity of flat slab construction with girder and slab construction.

In the interest of truth and fair play there is one correction that should be made in Table 15 given by Mr. Mensch. This table, which has been quoted several times in current literature, is based on one originally given by Mr. A. B. MacMillan, in which he computed the steel in the Turner slab for two different values of unit stress, *viz.:*

(*a*) at unit stress 16 000; weight = 549 lb. per panel;
(*b*) at unit stress 13 000; weight = 718 lb. per panel.

Mr. Turner explicitly stated that his formula, $\frac{W\,L}{50}$, was based on a unit stress of 13 000 lb., and Mr. MacMillan* made the same statement. The 718 lb. is consequently the only figure for which Mr. Turner would be responsible, and the weight of 549 lb. is entirely unauthorized, as his formula of $\frac{W\,L}{50}$ avowedly was based on a unit stress in the steel of 13 000 lb., as appears by reference to his book, page 29. For an assumed unit working stress of 16 000 lb., Mr. Turner's moment formula would become, in round numbers, $\frac{W\,L}{40}$.

* *Proceedings*, Nat. Assoc. Cement Users, Vol. VI, p. 266.

Mr. MacMillan further stated his ignorance respecting the amount of steel in the radial and ring rods of Turner's mushroom head, and assumed that their function was merely to support the slab rods in position at the top of the slab. The steel in the head would, in fact, be in the neighborhood of 200 lb., and it is evidently sufficient to add considerable stiffness over the columns, so that a conservative estimate of the steel per panel in Mr. Turner's construction would be 900 lb., instead of the figure wrongly stated by Mr. Mensch. This figure would be further increased by the usual laps over the column heads, so that substantial misrepresentation is made, intentional or not, by this garbled quotation. Mr. Mensch was himself present and contributed to the discussion of Mr. MacMillan's paper and has first-hand knowledge of these facts. It should be further noted that Mr. MacMillan's design, as shown by him in his paper, is almost identical with Mr. Turner's, with the omission of the inner ring rod, and his weight of steel only slightly exceeds Mr. Turner's design, as above corrected. Mr. Eddy.

He states* that:

"Floors designed by his method have been low in cost, have shown remarkable powers of resisting abuse, have stood tests of twice the live load over an entire bay, not only without signs of failure, but with such trivial deflection that one is led to believe that the constituent materials are far from being stressed inordinately."

Observe that this loading is not over one panel only but "over an entire bay," which disposes of Mr. Godfrey's contention that such loading would be fatal to this kind of structure.

It will be noticed that Mr. MacMillan's computation of the slab steel, according to Grashof's formula, provides nearly the same weight of steel as Mr. Turner uses, and would properly require additional steel to form a stiff head such as Messrs. Turner and MacMillan use. In round numbers, the weight of steel in either of these three slabs is, in all, 1 000 lb. per panel. These computations are really based on a flat plate theory of slab action in distinction from beam theory. The other four computed weights in the table are really based on beam theory, and require about twice as much steel, or about 2 000 lb. per panel. Beam theory practically neglects the co-action of the various belts, which co-action is considered and allowed for in slab theory. It affords a partial relief from the tensile stresses arising from the applied moments, a relief traceable to action of the bond shear of the embedment.

The statements that have frequently been made respecting the meaning of the results shown in this table seem to be misleading, as a correct interpretation of them is apparently to the effect that a beam arrangement, such as is involved for example in two-way rein-

* *Loc. cit.*, p. 266.

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forcement, requires perhaps twice as much steel as a slab which can develop true plate action.

The advocates of beam theory insist that any flat plate action that may be developed is uncertain and unreliable, and that slabs, in order to be safe, must be designed by beam theory.

The advocates of flat plate theory insist that innumerable tests have shown that flat plate action affords a perfectly reliable and dependable form of resistance when the design is properly made. The evidence for this proposition is regarded by them as perfectly conclusive, and they find it difficult, to say the least, to endure with patience the intimations which are continually made that flat slab design is essentialy unsafe unless it fulfills the requirements of beam theory. It would be just as logical to insist that a dome constructed so as to resist circumferential tensions must be designed on arch theory alone, and neglect circumferential action in the dome, as to insist that a flat slab, with a multiple-way reinforcement tied together by the embedment, must neglect the effect of that embedment.

Mr. Hatt is ready freely to admit the facts of which all who have made tests on buildings are perfectly aware, namely, that the observed stresses in the steel are actually about one-half as great as would be found by considering the steel to resist the entire applied moments.

If other critics have known the facts and have not been willing to acknowledge them, they have been disingenuous; if they have not known them, they were disqualified to discuss the subject by their ignorance of the essential basis of the discussion. Either alternative would seem to discount the value of the opinions which such critics have expressed, and would seem to make the basis of their opinions dubious.

When Mr. Hatt, however, refers to the tensile stresses in the concrete, which he says carry the remaining fraction of the applied forces, he evidently has in mind direct tensile stresses in concrete, acting parallel to the steel and to be added to those in the steel, such as would disappear and be ineffective whenever cracks appeared across these lines of stress. That is a naïve and quite natural explanation, and one which is evidently correct in the case of reinforced concrete beams, but can be shown to be incorrect in the case of slabs.

Owing to the stresses which take place in multiple-way reinforcement in slabs, the shearing stresses due to the bond between reinforcement and concrete play a rôle, and have a fundamental importance, that has not hitherto been realized or understood, and they produce effects such as they do not produce in beams. This has been explained elsewhere. Concrete stresses arising from bond shear may be correctly designated as indirect stresses, in distinction from those previously

mentioned as directly due to bending. They are operative and to be depended on as long as the bond remains intact, which is until final failure, and long after the cracks due to the direct stresses of bending have appeared. These indirect stresses are the ones which produce the effects Mr. Hatt recognizes. His judgment tells him that direct tensile stresses are unreliable. Agreed; but should he become convinced, however, that the concrete in combination with the steel in the slab affords a resistance of a reliable kind, as it does not in beams, he would be in a position to revise his estimates as to a correct basis for safety of design. Mr. Eddy.

In tests up to perhaps one-half of the design load it is usual for direct stresses in concrete to be of so much assistance to the reinforcement as to make the steel stresses comparatively small. For larger loads, up to two or three times the design load, or more, direct stresses cannot be relied on, but indirect stresses are predominant in the concrete. The true properties of reinforced concrete, as a combination which is sufficiently fine-grained to exhibit resistance such as might properly be ascribed to a single homogeneous material different from either steel or concrete, then appear. It is to this that the formulas of the book on Flat Slabs apply, as they do not to lower stresses. The material is one that is tough and not subject to sudden collapse or unexpected failure, and will show perfectly evident signs of distress long before gradually giving way. Such material must not be made with its bars too large or too far apart, as that makes it too coarse-grained and decreases the surface on which bond shear acts.

Comparatively few tests to destruction have been made in slabs, but those that have been made fully support the view here advanced. When, however, the experimental evidence shall become sufficient, Mr. Hatt will feel compelled, doubtless, to accept it as a fact, but the same can hardly be said of a number of those who constantly discuss slab theory in print, but who are so wedded to preconceived *a priori* conceptions as to how slabs must act as to be wholly incapacitated for any change of view, no matter what the facts may be. For reasons stated at the beginning of the paper, the writer disagrees with Mr. Hatt's opinion that the controlling factor in slab design should be the compressive stresses in the concrete rather than the tensile stresses in the steel. One reason for this is the uncertainty still existing as to the resistance which concrete or any other solid can offer to hydraulic compression. For example, does any engineer suppose for an instant that the rocks in the depths of the earth's crust, which are subject to enormous hydraulic stresses by reason of the superincumbent strata, are thereby crushed or rendered less able to resist shear or tension?

Mr. Eddy.

The condition of the concrete under compression in a slab is not comparable to that in a test block or cylinder in a compression test, so that the uncertainties respecting the concrete in a slab are greater, if possible, than those respecting the steel, and the measured compressive stresses in the concrete, on which Mr. Hatt would rely, furnish no reliable criterion, for it is highly probable that shearing rather than compressive stresses are the determining factors in the ultimate resistance of the concrete.

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ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion.*

By Messrs. William de H. Washington, Willis Whited,† William H. Connell,† P. E. Green,† A. F. Masury, and W. R. Farrington.†

ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

Mr. Washington.

William de H. Washington, Assoc. M. Am. Soc. C. E.—In connection with the discussion of the subject of highway engineering organizations, the members of the Society are referred to a report to the Hon. John N. Carlisle, Commissioner of Highways of the State of New York, by the Board of Consulting Engineers consisting of Harold Parker and George C. Diehl, Members, Am. Soc. C. E., and the speaker. In addition to covering specifications for various types of roads and pavements and the materials to be used therein, the report considers the subject of efficient organization for the State Highway Department of New York, and gives in detail the work to be covered by such an organization, the duties of the various engineers included in the organization, and charts covering the interrelationship of its component parts. The report is on file in the Library of the Society.

Mr. Whited.

Willis Whited, M. Am. Soc. C. E. (by letter).—In reply to the very able and interesting discussion which has been elicited on this subject, the writer would call attention to the following points:

Mr. James includes Pennsylvania among the States in which road taxes are paid in labor. This was abolished by an Act of Assembly in 1911, and all road taxes are now paid in money.

Much is said on the subject of the construction and maintenance of improved roads, that is, roads covered with some kind of paving. More than 90% of the roads in the United States are not improved, in

* Continued from March, 1914, *Proceedings*.
† Closing discussions.

Mr. Whited.

the sense mentioned, and probably will not be for many years to come. The most important problem that confronts the State highway administration, is to make these roads as good as possible at a minimum expense, and generally with a view to more permanent improvement at some future time. If an engineer has work to do that is worth $10 000, and has $10 000 with which to do it, the proposition is comparatively simple, but if he has $10 000 worth of work with only $1 000 with which to do it, a very high grade of engineering ability is called for.

Engineering is a union of common sense and science. It deals with the forces of Nature; but by far the most difficult force with which the engineer has to deal is human nature, and ability to deal with human nature can hardly be acquired in any college course. It requires genius, to begin with, and proper training, which includes a good knowledge of the fundamental principles, including moral principles, and the ability to apply them effectively.

Mr. Durham's remarks bring out strongly the fact that although the organization in Paris is almost ideal, the results are open to criticism; whereas, in London, notwithstanding the fact that the organization contains obvious defects, the results are excellent; and in Berlin where everything is under the control of a Government which has a genius for organization, the results are variable. These facts point strongly to the indication that administration is a personal matter, rather than a question of form of organization, and, to produce the desired result, it must be sufficiently elastic to permit of all men being assigned to the duties for which they are best adapted.

The French are very systematic and competent, technically, but inferior to the English in executive ability. The Germans are very skillful and industrious, but are so accustomed to military rule that difficulty is apt to develop in case of an emergency requiring special methods.

The writer is often inclined to deplore the position of many in the Profession who advocate the placing of engineers at the head of all work that is even partly of an engineering nature. The combination of a high grade of technical skill and knowledge with the sure and accurate judgment of men that is required in any high administrative position, together with the proper understanding of the many legal, commercial, and political questions involved in State highway administration, is so rare as hardly to be looked for on this earth; and, for a man at the head of such an organization to be notably deficient in any of the latter requirements is almost certain to wreck the department; whereas, a man possessed of these qualifications, as well as the self-control and tact required for such a position, can usually make a success of it, even though he is equipped with but a moderate amount of technical knowledge. A man possessed of the latter, with a sufficiently elastic

Mr. Whited.

organization, will have the power and the disposition to avail himself of any special knowledge or skill possessed by any member of his staff, whatever his title may be, in the solution of any problem that may present itself. Every man has his weaknesses and deficiencies, and it is of the utmost importance that the "Chief" see that none of them is allowed to interfere with the smooth running of the organization.

The writer would suggest that the most important qualifications of the head of any department of public works are tact, common sense, self-control, and the ability to deal with men, both personally and commercially, and a knowledge of his legal rights and duties, as well as those of the traveling public, the public service corporations, and the abutting property owners. The more knowledge of engineering technology he has the better, but the questions which arise in strictly administrative work are generally questions which must be settled on the spot, while those of a technical nature can usually be referred to some expert in paving, bridge construction, tests of materials, etc., who should be included in the staff of the department.

A word as to the politician: The general idea is that the politician is a professional criminal, whereas, the fact is that politicians of the higher grade are fully as honest as men in most other lines of business, and are not only possessed of tact and common sense, with a thorough knowledge of the world, but are also industrious as a rule.

As to the statement by Mr. Bennett, to the effect that a prominent engineer has said that there were few engineering problems to be faced in highway work, the writer would say that a good definition of an engineer is, a man who can do with one dollar, what any one can do with two; but, considering the many millions of dollars spent annually on road construction and maintenance, and the importance of the interests involved, there is ample scope for the exercise of the best grade of engineering ability.

It is doubtful whether anybody has reached, or ever will reach, a final settlement of the conflict between the inspector and the contractor's foreman, at least in the present state of human nature. The strictly competent inspector does not exist, and never will, and the same may be said of every one connected with the work. The only thing to do is to fight out each individual question as it arises. The writer will suggest, however, that a frequent cause of the differences between the contractor's foreman and the inspector of small and limited experience, is that the foreman, or some of his workmen, with or without the knowledge of his superiors, will in some way get the better of the inspector when he is off his guard, and the inspector finds it out when it is too late. The anger, suspicion, and hostility which takes possession of the inspector need not be explained to any one understanding human nature. It is easy to deceive anybody, but it is almost certain that it will be found out eventually.

Mr. Whited. Regarding efficiency boards, the writer would say that the efficiency expert has his place in all public or private work, but his outgivings can seldom be taken as final, because so much depends on the personality of the officials and employees, both high and low.

Mr. Connell. WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E. (by letter).—The discussions on "Municipal Highway Engineering Organizations" would seem to indicate that there is a general tendency to raise the standard of organizations controlling this important branch of engineering work. The exceptions taken to the organization advocated by the writer are matters of minor detail, and in some cases are due to a misunderstanding. A few of those who discuss the subject are apparently under the impression that the writer advocated a standard highway organization. This, of course, is not the case, as such an organization would not be possible on account of the varying sizes of the municipalities; but an attempt was made to outline the general principles which could be followed in forming an organization suitable for municipalities of various sizes. Some seem to dwell at length on coping with existing conditions. That, of course, is necessary, but is simply routine; it would not do, under any conditions, to plan an ideal organization which could not be operated successfully under different laws, ordinances, and existing conditions, over which no control could be exercised; and, therefore, the first thing to do is to plan the work in such a way that it can be carried on successfully and economically under existing conditions, and try to have these conditions improved so that the ideal organization can be put into operation in the course of time.

One of the questions of detail, on which there seems to be some difference of opinion, is whether there should be a separate organization for maintenance. It would seem to be logical that the organization to take charge of the maintenance should be the one under which the pavements were constructed, as the men in that department understand all the conditions in connection with that work—some of which might at a later date be factors in the maintenance problem. Having a single organization also does away with the shifting of responsibility. The maintenance is just as important a part of the road problem as the construction, and no pavement should be laid without taking into consideration, among other things, its probable life and the probable maintenance charges during its life.

Another very important consideration, in having one organization in charge of both branches of the work, is that it tends to build up a better personnel and makes the men more generally useful. In other words, an important part of the education of the highway engineer is to understand and realize the importance of the maintenance problem. A good many of them do not realize the importance of this problem

Mr. Connell.

because, in many cases, separate divisions are in charge of the construction and maintenance, and the men who have nothing to do with maintenance, as a rule, devote very little time and study to the subject. There is no doubt that more study would be given to the general highway problem if all the work was under one organization; the engineers who constructed the pavements would know that they were to be held responsible for the maintenance, and would naturally give more time and thought to this phase of the work. Aside from this fact, it would require a smaller organization if combined than if separate, as each division—no matter how well organized—tends to a duplication of work, to a certain degree.

One member advocates the general scheme of organization, but points out that the weak point of the system is in the question of personnel, and further states that in the United States there is no trained class of public works officials or Government engineers, organized like the French corps for bridges and highways. In view of this, he states that, in the light of possible conditions, it would seem to be better policy to concentrate along all classes of work, organizing the bureau in sections for doing specific things, in charge of the man best qualified to do them. This, of course, is a proposed scheme to cope with existing conditions, which is just what should be avoided. Engineers are often too apt to accept conditions as they are, instead of trying to improve them, and if they do not strive to form an organization that will stand and that is not dependent on any one man suited to fill a particular position, the organization can never be a stable one. A highway organization should not be dependent on any individual, but should be designed so that it would be constantly training men to fill the vacancies of those passing out, and it would appear to be better policy to endeavor to change the conditions that prevent formulating an organization of this kind, than to become reconciled to existing conditions.

A very important consideration in connection with the highway organization is the question of the *esprit de corps*. Mr. Meeker states that the key-note of all successful organizations is that all must work harmoniously. No matter how well the corps is organized, without harmony it will be a failure. This is a very important matter, and cannot be given too much attention as a factor in the control of a good working organization for this or work of any other class.

There was some discussion regarding the method of handling the patrol inspection. The writer advocated that the street-cleaning force should be combined with the general highway organization, and that the maintenance patrol inspection of the pavements should be handled by the street-cleaning inspectors or the foremen. The advisability of this method was questioned. This was advocated in order to increase

Mr. Connell.

the usefulness of the men in charge of the inspection work, and thus reduce the cost of the patrol inspection by utilizing men already on the work, such as the street-cleaning inspection force, instead of employing additional men to cover the ground every day. There is no reason why the police could not also be utilized for work of this kind, and when our municipalities are further advanced, and more on a business basis, there is no doubt that they will be more generally used in connection with matters of this kind than at the present day.

If some general principles were formulated relative, not only to the character of the organization, but the details of conducting the work, resulting from discussions of this kind, they would carry a certain weight and would be a great help to engineers throughout the country in having the laws, regulations, and ordinances changed to make it possible to plan an organization in accordance with what is conceded to be the best practice by the authorities on this subject. In other words, it is always easier to put into effect anything that is the combined opinion of the engineers of the country at large than it is to accomplish it by individual effort.

FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

Mr. Washington.

WILLIAM DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—Based on experience as a contractor and engineer, the speaker wishes to emphasize the importance which he attaches to traffic censuses in connection with this topic. In this matter, much may be learned from European engineers, as traffic censuses have been taken for many years in both France and England. Such censuses, and soil and geological surveys, should form an important part of the work preliminary to the design of any highway.

During its investigations in New York State, Commissioner Carlisle's Board of Consulting Engineers,* realizing the variety of local conditions obtaining in different parts of the State, recommended a "material" survey of the State, in order to make available to the Highway Department information of inestimable value if local materials were to be used economically.

As an example of proper utilization of local material might be cited the case of a section of Route 4, extending across the southern tier of counties of New York. It was found that an appropriation of $1 000 000 was available for the construction of more than 150 miles of road. In order to meet the demands of the people for a maximum amount of mileage to be constructed under this appropriation, the engineers had designed a single-course semi-Telford road. The local material available was a soft sandstone which would rapidly disin-

* Report of the Board of Consulting Engineers, 1913.

tegrate under wear and weather. Due to long hauls from railroad stations, it was found that the importation of limestone for the base and trap rock for the top would require the expenditure of a large sum of money. The Board of Consulting Engineers, therefore, decided to construct a cement concrete top, using for the mineral aggregate the local sandstone, on a base of sandstone, and to complete the pavement with a bituminous carpet. Mr. Washington.

Highway engineers require more definite knowledge relative to the life of various types of roads when subjected to varying local conditions. Massachusetts, apparently, is about the only State which has made a careful series of records covering the wearing qualities of certain classes of materials and types of roads under given traffic conditions. Every State should emulate Massachusetts and endeavor to ascertain, under their local conditions, the value of the materials which are available for road building and the service which a given type of road will give under known traffic, and careful records should be taken covering the rate of wear under traffic and the deterioration due to various climatic conditions.

Mr. Crosby has mentioned the subject of harmonic action and harmonic waves in road surfaces. This expression was evolved by that able engineer and road builder, Col. R. E. Crompton, of London, who has ascertained that roads have a tendency to creep or to form miniature hills and hollows. With certain types of traffic, he has found that the distances between the hills and hollows have a certain relative length. The speaker, in company with Col. Crompton, examined one road on which there were more than 1 000 motor busses per day, and found a series of long waves, which occurred with great regularity throughout the length of the road. The speaker believes that improper rolling of the road and the compression of the material are the main factors influencing the formation of harmonic waves. Traffic further develops the harmonic waves due primarily to the vibration of the motor cars passing over them. During the process of rolling it will be noticed that the material rises in front of the front roller and pushes ahead to a certain extent, finally ceasing to move forward, at which time the roller goes over it. If examined, the road surface will be found to have a marked swell. Col. Crompton has endeavored to solve this problem by devising a machine with three sets of rollers, instead of the ordinary tandem roller. The first and third rollers are somewhat lighter than the second one. The first roller consequently smooths and compresses the material, but does not put excessive weight on it. The second and third rollers give the additional compression necessary. It has been found possible to secure a much smoother base and surface for the road with this triple roller than with the ordinary tandem roller.

Mr. Green.

P. E. GREEN, M. AM. SOC. C. E. (by letter).—It is gratifying to find such a general agreement as to the main points among the eminent engineers who have engaged in this discussion. Naturally, there have been some sharp disagreements, but these have been rather few. Apparently, road construction and maintenance are becoming standardized, a result to be desired in many ways.

Mr. Whinery seems to object mildly to the suggestion that the esthetic features of the highway should be considered, but it is believed that his objections are not well taken. It is so easy to consider them, the result is so desirable—often at little or no additional expense—that it would appear poor policy, both from a utilitarian and artistic point of view, to ignore such benefits. Too often engineers are justly charged with lack of appreciation of that very point.

It is believed also that Mr. Whinery is incorrect in his statement that the temporary and inadequate railways built in the early days resulted from the undeveloped conditions of the principles and requirements involved, rather than from financial or economic considerations. Undoubtedly, much poor engineering was perpetuated, but also much of it was of the highest order, in that it adapted its construction to its pocketbook. It is comparatively simple to build well with an unlimited pocketbook, but certainly a higher order of intelligence is often required to do well with little money.

As to the roadway diagram, Fig. 1, it would appear as if Mr. Whinery failed to use it with the discretion advocated by the writer. According to the diagram, the roadway width of the boulevard having 600 vehicles per foot of width should have been at least 80 ft., instead of 38 ft., as it actually is, and, if future traffic increase is taken into account (as it should be), the width would be increased accordingly. It cannot be maintained that the roadway width is a straight-line function of the traffic. It is believed Mr. Whinery would think better of the diagram in question if he studied it and its text a little more carefully.

Mr. Tillson encountered the same obstacle that the writer found; that is, the difficulty in describing "new matter", but he got around it more gracefully. However, the writer disclaims its authorship; it was simply one of the Society's rules governing this discussion.

Mr. Tillson further calls attention to the late practice of the City of Paris in treating paving blocks with oil under pressure to the amount of 10 or 12 lb. per cu. ft. This is largely for the purpose of preventing the blocks from swelling. The writer, however, believes that swelling will not be prevented in that manner. At Longview, Tex., in September, 1913, after a hot dry summer, it was found that blocks treated with true creosote oil in 1911 absorbed nearly 50% of their volume of water and had lost nearly 50% of the contained oil.*

**Engineering News*, December 4th, 1913.

It is believed that there is only one way to treat wood blocks permanently so that they will not absorb a great deal of water, and that is by traffic. Mr. Green.

The writer thoroughly agrees with Mr. Meeker and Mr. Washington that the so-called "harmonic" waves are caused more by poor workmanship and materials than anything else. On Diversey Boulevard, Chicago, paved with bituminous concrete in 1911, at a section having a mixed traffic of about 5 000 vehicles per day, evenly divided as to direction, one side has pushed into these waves so that it has become exceedingly rough. The other side is quite smooth. This condition is apparent for only about 2 000 ft. The rest of the highway (several miles) is in perfect condition. It is very evident that defective material or workmanship or both has been the cause of this.

EQUIPMENT AND METHODS FOR MAINTAINING BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

Mr. Washington.

WILLIAM DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—Three members of this Society were engaged by the Highway Department of the State of New York to devise methods of maintaining highways and reducing the cost of maintenance. They concluded that the railroads had set a very excellent example as to the organization for upkeep, by which they put this work in the hands of a body of men under a competent foreman, instead of distributing their men individually over a section. These men work as a unit. Each group of men is provided with all necessary materials, which materials are usually carried on hand cars.

The speaker has devised a machine to accomplish exactly the same object in connection with the maintenance of roads, and meets the requirements mentioned in Mr. Farrington's paper. This movable repair plant consists of a motor truck equipped with all tools and machines necessary for the maintenance of highways, and supplied with storage capacity for broken stone, sand, and tar. The machines include a pressure spraying device, sand and stone chip distributor, a roller (beneath the body of the truck), a stone and sand dryer, a mixing machine, a road sweeper, a revolving power scarifier, extension arms for pulling a plow, scraper, split-log drag, or a snow plow, a power loader and derrick, and all requisite small tools. The truck has storage capacity for about 4 tons of broken stone and from 12 to 14 bbl. of tar. When used for the repair of macadam roads, a large removable water tank is carried on the truck.

The outfit has been designed with the intention of utilizing it in the maintenance of all types of roads and pavements constructed by the State Highway Department of New York.

Mr. Masury.

A. F. MASURY, ESQ. (by letter).—There is no equipment better adapted for the maintenance of bituminous surfaces and bituminous pavements, or for making roads, than motor trucks; and probably none is more generally in use for this purpose.

This discussion will give facts, based on the actual operation and maintenance of trucks, which will enable the engineer and contractor to operate them to the best advantage in road work.

Factors of Motor Truck Operation.—Glaring generalities have been characteristic of most discussions on motor truck operation. The motor truck salesman, with his optimistic figures and his aggressiveness to make a sale, has caused the engineer, contractor, and merchant to use his figures in planning truck operation. This, done without much thought on the part of the owner, really gives him the salesman's figures of past performances on which to base his intended operation.

The commercial motor vehicles are machines, made in a variety of types and sizes, designed for specific purposes, and must be built, their field of operation laid out, their operation controlled, and the upkeep looked after, by engineers.

The purpose of this paper is to present data on different sizes and types of motor trucks, to show actual figures as to their performance, and state the factors which must be used in operating them.

Arrangement of Space for Most Economical Storage.—In storing motor trucks, it has been worked out in practice that they can be handled best in a building of usual size by arranging them in two rows with an open aisle between; that is, the trucks are backed into place, with their fronts facing the aisle. This permits them to be taken out without any delay, and also allows them to be run into the aisle, where washing and drainage facilities should be provided.

In most buildings, posts are necessary, and should be toward the front of the trucks, but far enough back to allow the trucks to turn out into the aisle. The depth from the wall to the post should be at least 16 ft. 6 in.; thus, the front wheels of trucks of most sizes will project beyond the posts and allow an easy and quick turning into the aisle. The aisle should be about 27 ft. wide in order to provide a practical turning radius for getting out or backing in. The posts should be about 22 ft. apart, so as to allow the storage of three large trucks between them. Where a large number are stored the proportioning of small to large trucks allows this arrangement to be worked out with the economical use of all space.

The size of the doors should be looked into very carefully, as trucks, especially of the van type, are now very large. In order to take those of the largest type, such as vans and those used for crackers

and paper boxes, the doors should have a clearance of at least 12 ft. 3 in., and should be double, opening outward. The clearance between the butts of the doors should be at least 10 ft. 9 in. This allows sufficient space for the fastenings of the doors, without interfering with the trucks as they move in and out. There should be lights on the butts of the doors on each side, quite high up. Mr. Masury.

The elevators must also be considered carefully. They must be at least 24 ft. long and 11 ft. wide, in order to take the modern motor truck, and should have a carrying capacity of at least 7 tons, in order to accommodate the trucks when light. A heavier elevator, to take care of loaded trucks, is impractical, from a commercial standpoint; loaded trucks are not usually garaged in such buildings, but might be accommodated on the ground floor. The elevator should have four guiding rails, and a speed of about 60 ft. per min.

Service.—There must be adequate facilities for filling the tanks with gasoline, and oil, and for the storage of the large quantities of lubricants used in the operation of such trucks.

To comply with fire prevention regulations, the gasoline must be stored underground, and must be piped to at least two outlets on each garage floor. There are two practical ways of raising it to each floor: by hydraulic power and by pressure. Either of these systems can also be used for cylinder oil and kerosene. There should be at least one outlet for kerosene and one for oil on each floor. Meters should be attached to all outlets, and provision should be made for locking them. At least one portable tank should be provided for each floor, as trucks must be filled during the hours when the garage is most crowded, and when, at best, only two can be filled from each outlet.

Oil and kerosene may be carried to the trucks from the outlets in measures, as comparatively small quantities are used.

Transmission oil, cup grease, wood alcohol (for use in the radiators in cold weather), and other material and accessories may be stored in a room on the floor with the trucks.

There should be an efficient system for keeping account of the exact quantities of supplies consumed by each machine.

A dirty truck (and trucks collect dirt with astonishing rapidity) is much harder to keep in good order than a clean one, as dirt not only gets between moving parts and into bearings and causes wear, but it prevents the discovery of minor troubles until they have become major ones, and makes repairs a difficult and distasteful job if they are discovered. Therefore, trucks should be washed at night. For this purpose a regular force should be employed, its size depending on the number of cars to be washed. Four washers and two polishers can take care of fifty cars.

Mr. Masury.

Drivers are not always to be depended on to make repairs; their hours are long, and they have little time for them during the day's run. For this reason, if six or more trucks are being operated, it will be well to have a night repair man on duty, with one or two helpers. As the trucks come in, each driver should report any trouble with his machine. The repair man can also examine the trucks to discover loose bolts, and can grease them up.

Truck Sizes.—Trucks may be divided into three classes, according to the arrangement of the motor and the driver's seat:

Type 1.—This is the usual touring car type; the motor is over the front axle, the driver and control are behind the motor.

Type 2.—In this type the motor is over the axle; the driver and control are directly above the motor.

Type 3.—The motor is over the front axle; the driver and control are on one side of the motor, and the helper is on the other side.

Types 2 and 3 conserve the length of the chassis, as less space is taken for the driver and control, thus allowing a longer loading platform in proportion to the wheel base than Type 1.

The most important average dimensions for each size of truck are given in Table 2; and the diameters of turning circles are given in Table 4.

TABLE 2.—MOST IMPORTANT AVERAGE DIMENSIONS OF TRUCKS.

Type.	Capacity.	Wheel base.	Extreme length.	Length, back of cab.	Extreme width, in inches.	Height of Loading Platform, in Inches.		Clearance, in inches.
						Empty.	Loaded.	
Type 1	1-ton.	12 ft.	16 ft.	9 ft. 2 in.	68	36	32	9½
	2-ton.	13 ft.	17 ft.	9 ft. 2 in.	68	38	34	9½
	3-ton.	13 ft. 6 in.	20 ft. 6 in.	11 ft. 0 in. to 11 ft. 6 in.	87	42	38	10
	4-ton.	14 ft. 6 in.	21 ft. 6 in.	12 ft. 0 in. to 12 ft. 6 in.	87	42	38	10
	5-ton.	15 ft. 6 in.	22 ft. 6 in.	13 ft. 0 in. to 13 ft. 6 in.	87	44	38	10
	7½-ton	16 ft 6 in.	23 ft. 6 in.	14 ft. 0 in. to 14 ft. 6 in.	87	44	38	10
Type 2	3-ton.	11 ft. 6 in.	18 ft. 3 in.	12 ft. 6 in.	87	42	38	10
	4-ton.	12 ft. 6 in.	30 ft. 3 in.	14 ft. 6 in.	87	42	38	10
	5-ton.	13 ft. 6 in.	22 ft. 3 in.	16 ft. 6 in.	87	44	38	10
	7½-ton.	14 ft.	23 ft. 3 in.	17 ft. 6 in.	87	44	38	10
Type 3	5-ton.	11 ft. 10½ in.	18 ft. 1 in.	14 ft.	93½	46	40	7½
	7-ton.	13 ft. 4½ in.	20 ft. 1 in.	16 ft.	93½	46	40	7½
	10-ton	14 ft. 2½ in.	22 ft. 1 in.	18 ft.	93½	46	40	7½
Power-dump truck, Type 1.	5-ton.	11 ft. 9 in.	16 ft. 5 in.	8 ft. 3 in.	87	44	39	10½
	7½-ton.	12 ft. 10 in.	17 ft. 5 in.	9 ft. 3 in.	87	44	39	10½

Factors in Cost of Operation of Motor Trucks.—Table 3 shows the cost of operation of trucks of standard capacities running at different mileages per day, and the factors in detail of each item of upkeep. These figures are taken from actual costs of operating trucks, and are nominal in every way. Interest is reckoned at 5% on half the cost of the truck; insurance at 2% on 80% of the cost; and amortization at 10 per cent. Mr. Masury.

Column letter in Table 3.		Formulas.
A	Capacity.	
B	Average cost of chassis, based on list prices of the number of manufacturers indicated by the figures in parentheses.	
D	Miles per hour.	
F	Driver's salary, per year.	
G	Cost of garaging, per month.	
H	Miles per day.	
I	Maintenance (small repairs, etc.), per year.	
J	Gasoline, per year.	
K	Oil and grease, per year.	
L	Tires, per year.	
M	Interest, per life.	$M = R\frac{B}{2} \times 5$ per cent.
N	Insurance, per life.	$N = R \times (80\%$ of $B) \times 2$ per cent.
O	Depreciation, per life.	$O = \frac{B}{10\% \text{ of } R}$.
P	Cost, per year.	$P = F + 12G + I + J + K + L + \frac{M}{R} + \frac{N}{R} + \left(O = \frac{B}{10}\right)$.
Q	Miles of life at each mileage per day truck is assumed to run.	
R	Years of life.	$R = \frac{Q}{W}$.
S	Cost, per life.	$S = P \times R$.
T	Cost, per day.	$T = \frac{P}{300}$.
U	Cost of tires, per mile.	$U = \frac{L}{W}$.
V	Cost of gasoline, per mile.	$V = \frac{J}{W}$.
W	Miles per year.	$W = H \times 300$.

Mr. Masury.

TABLE 3.—FACTORS IN THE COST OF OPERATION OF MOTOR TRUCKS.

A	B	D	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W
Capacity.	Average cost of chassis.	Miles per hour.	Driver's salary, per year.	Garage, per month.	Miles per day.	Maintenance, per year.	Gasoline, per year.	Oil and grease, per year.	Tires, per year.	Interest, per life.	Insurance, per life.	Depreciation during life.	Cost, per year.	Miles of life at each mileage per year.	Life of truck, in years.	Cost, per life.	Cost, per day.	Tire cost per mile.	Gasoline, cost per mile.	Miles of use, per year.
1-ton	$1 963 (12)	19	$780 880 900	20	40 60 90	$120 240 360	$300 450 675	$120 190 270	$360 540 810	$408.75 231.15 117.76	$200.70 147.00 75.00	$1 626 839 426	$2 197.78 3 106.78 3 531.78	100 000 85 000 65 000	8.3 4.7 2.4	$18 241.74 14 601.86 8 476.27	$7.33 10.36 11.77	$0.03 0.03 0.03	$0.025 0.025 0.025	12 000 18 000 27 000
1½-ton....	2 225 (12)	18	780 880 900	21	35 50 90	135 270 405	270 395 710	103 150 270	841 488 878	598.97 317.06 193.50	881.00 269.00 85.00	2 140 1 270 595	3 109.73 2 098.73 3 728.73	100 000 85 000 65 000	9.6 5.7 2.4	21 117.41 15 929.51 8 948.95	7.33 9.00 12.49	0.0325 0.0325 0.0325	0.0262 0.0262 0.0262	10 500 15 000 27 000
2-ton	2 918 (8)	17	750 880 900	22	35 50 80	150 300 450	288 415 660	75 150 225	398 563 900	668.88 406.52 246.96	401.00 283.00 196.00	2 660 1 818 789	2 861.44 3 067.44 4 026.44	95 000 85 000 65 000	9.1 5.6 2.7	21 469.10 17 177.66 10 877.39	7.67 10.22 13.42	0.0375 0.0375 0.0375	0.0274 0.0276 0.0275	10 500 15 000 24 000
3-ton	3 575 (6)	15	780 880 900	23	50 50 80	180 360 540	395 487 781	75 150 225	678 1 081	898.75 473.68 241.31	572.00 303.00 154.00	3 575 2 269 1 018	3 223.39 3 612.39 4 939.39	90 000 80 000 65 000	10.0 5.3 2.7	22 333.90 19 145.67 11 716.35	7.44 12.04 14.49	0.045 0.045 0.045	0.0324 0.0324 0.0324	9 000 15 000 24 000
4-ton	4 066 (8)	14	780 880 900	24	50 50 70	240 480 720	360 600 840	150 225	750 1 025	538.94 364.95	720.00 311.00 195.00	4 500 2 980 1 210	2 920.50 4 072.50 4 566.31	90 000 80 000 65 000	10.0 5.3 3.0	29 205.00 19 451.44 18 698.08	9.74 12.32 15.22	0.06 0.05 0.05	0.045 0.04 0.04	9 000 15 000 21 000
5-ton	4 500 (10)	12	900 1 000	25	30 50 70	210 480 720	406 678 948	90 180 262	540 900 1 260	195.00 596.25 387.50	720.00 311.00 216.00	4 500 2 980 1 350	2 920.50 4 072.50 5 124.50	90 000 80 000 65 000	10.0 5.3 3.0	29 205.00 21 184.25 15 978.50	9.74 13.58 17.08	0.06 0.06 0.06	0.045 0.045 0.045	9 000 15 000 21 000
7½-ton....	5 400 (2)	0	800 900	26	25 40 510	255 510	395 690	94 150	750 1 200	1 525.50 887.00	976.00 535.00	5 400 3 348	3 937.40 4 403.40	85 000 75 000	11.3 6.2	38 051.69 27 073.06	11.22 14.88	0.10 0.10	0.0326 0.0326	7 500 12 000
10-ton	5 500 (1)	7	800 900	28	25 40 540	270 540	468 750	94 150	1 200 1 920	1 553.75 852.50	994.00 545.00	5 500 3 410	3 948.50 5 971.50	85 000 75 000	11.3 6.2	44 561.55 33 808.30	18.15 17.90	0.16 0.16	0.0324 0.0625	7 500 12 000

TABLE 4.—TURNING CIRCLES.

Mr. Masury.

Type.	Capacity.	Wheel base.	Diameter of turning circle, in feet.	Type.	Capacity.	Wheel base.	Diameter of turning circle, in feet.
Type 1...	1-ton....	10 ft. 6 in.	47	Type 1...	5-ton....	12 ft. 6 in.	57
		11 ft. 6 in.	53			13 ft. 6 in.	62
		12 ft. 6 in.	55			14 ft. 6 in.	68
Type 1...	1½-ton..	10 ft. 6 in.	47			12 ft. 6 in.	57
		11 ft. 6 in.	53	Type 1...	7½-ton..	13 ft. 6 in.	62
		12 ft. 6 in.	55			14 ft. 6 in.	68
		13 ft. 6 in.	60			15 ft. 6 in.	71
Type 1...	2-ton....	10 ft. 6 in.	47	Type 2...	3-ton....	11 ft.	47
		11 ft. 6 in.	53			11 ft. 6 in.	49
		12 ft. 6 in.	55			12 ft.	51
		13 ft. 6 in.	60	Type 2...	5-ton.....	11 ft. 6 in.	53
		12 ft. 6 in.	55				
Type 1...	3-ton....	13 ft. 6 in.	59	Tractor..	3-ton.....	10 ft. 4 in.	40
		14 ft. 6 in.	61	Tractor..	5-ton.....	10 ft. 8 in.	41
		12 ft. 6 in.	55				
Type 1...	4-ton....	13 ft. 6 in.	59	The trailer will usually cut under.			
		14 ft. 5 in.	61				

TABLE 5.—TIRE EQUIPMENT FOR TRUCKS.

(1) Truck Capacity.	(2) Axle load.	(3) Size of tires.	(4) $\frac{L}{W \times C}$	(5) Speed, in miles per hour.	(6) Mileage.	(7) Consumer's price.
1-ton.....	Front 2 600	36-4 single	18	16.5	7 000	$ 54.15
	Rear 5 200	36-5 single	28.8			70.45
	Rear 5 200	36-2½ dual	28.8			51.30
1½-ton.....	Front 2 800	36-4 single	19.4	15	7 000	54.15
	Rear 6 300	36-3 dual	29.2			68.80
2-ton.....	Front 3 060	36-4 single	20.8	14	7 000	54.15
	Rear 7 200	36-3½ dual	28.6			83.67
3-ton, Type 1.....	Front 4 300	36-5 single	23.9	12½	7 000	70.45
	Rear 10 800	36-4 dual	37.5			108.35
	Rear 10 800	42-4 dual	42.1			126.17
3-ton, Type 2.....	Front 5 950	36-5 single	33.0	12½	7 000	70.45
	Rear 9 600	36-4 dual	33.3			108.35
	Rear 9 600	42-4 dual	28.5			126.17
4-ton.....	Front 4 950	36-6 single	22.9	11	7 000	87.97
	Rear 12 950	36-5 dual	36.0			138.90
	Rear 12 950	42-5 dual	30.8			172.40
5-ton, Type 1.....	Front 5 300	36-5 single	29.4	10	7 000	70.45
	Rear 15 200	36-5 dual	42.2			138.90
	Rear 15 200	42-5 dual	36.2			178.40
5-ton, Type 2.....	Front 7 425	36-6 single	34.4	10	7 000	87.97
	Rear 15 425	36-5 dual	42.8			138.90
	Rear 15 425	42-5 dual	36.7			172.40
7½-ton.....	Front 6 175	36-6 single	28.6	7½	7 000	87.47
	Rear 20 325	36-6 dual	49.3			175.97
	Rear 20 325	42-6 dual	42.2			205.85

In Table 3 the cost of the body is left out intentionally, as it usually represents the amount of discount given by the manufacturer, and it needs but few repairs.

Tire Data.—The data on tire equipment in Table 5 show the average of the requirements and guaranties of the different tire man-

Mr. Masury.

ufacturers, and also the average cost of tires to the consumer. Column 2, the axle load, gives the average weight on the front and rear axle of trucks of each size. Column 3, the size of tire, shows the sizes usually recommended by tire manufacturers for the respective loads carried on each axle. Column 4, $\frac{L}{W \times C}$, means the load on the axle, in pounds, divided by the width of the tire (the width being taken at the place where the tire meets the ground); C is the circumference of the wheel. The figures in this column are pounds.

This factor takes into account the size of the wheel, and therefore reconciles the fact that a larger wheel can have a tire of a smaller size, and is a better factor to use in comparing tire sizes than the weight, in pounds per square inch, of tire in contact with road.

Column 5 shows the normal speed of trucks of each capacity. Column 6 shows the guaranteed mileage by the tire manufacturers. Column 7 shows the average price at which tires can be bought by the consumer.

Mr. Farrington.

W. R. FARRINGTON, M. AM. SOC. C. E. (by letter).—The writer notes that considerable attention has been given by those taking part in the discussion on "Methods and Equipment for Maintaining Bituminous Surfaces and Bituminous Pavements" to the use of motor equipment.

Not much has been done, as yet, with trucks in State highway maintenance work in Massachusetts. Trucks have been used in some cases, but, except for transporting broken stone, gravel, bituminous materials, etc., over long hauls, apparently have not demonstrated their economy under existing conditions. The writer has no doubt, however, that, as this work increases and longer sections come under maintenance, motor trucks with the necessary equipment for patching, etc., will prove economical and will come into general use.

As to the material truck described by Mr. Washington, though the writer has no doubt that this will be useful, there would seem to be some objection to its general adoption at present, as it is probable that many of the culverts and bridges, even on main lines, are not strong enough to carry a truck of this weight, and there is also some question as to the effect on the road surfaces.

Recognizing the injurious effect of very heavy trucks on roads, where the surfaces and foundations, as well as culverts and bridges, are not especially designed for such traffic, the Massachusetts Legislature in 1913 passed a law prohibiting the use on the highways of trucks weighing more than 14 tons, without a permit from the State, city, or town officials having charge of repairs on the highways, and it would not seem to be good policy for the State to use on its work trucks of greater weight than the public generally is allowed to use.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

By Richard R. Lyman, Assoc. M. Am. Soc. C. E.†

Mr. Lyman.

Richard R. Lyman, Assoc. M. Am. Soc. C. E. (by letter).—Further thought and study, and the matter presented in the discussions on this paper, have confirmed the writer's conviction that the weir without end contractions is the weir which, in actual practice, will measure water in the most satisfactory way. When it is used, the diagrams‡ and tables presented in this paper give results, without computation, in accurate accord with existing experimental data. They give results, too, for a great variety of weirs, covering a wide range of heads, without that "troublesome correction" for velocity of approach.

Table Giving Discharges for all Weirs Without End Contractions, Within the Limits of Experimental Data.—For some purposes, tables giving discharges are more convenient than diagrams, therefore, discharges for a series of definite heads are presented in Table 69. Discharges for heads between those given may be found by interpolation.

Table Based on Formulas, Giving Discharges for High Heads.—A table of discharges, for heads higher than those for which accurate experimental data exist, by Gardner S. Williams, M. Am. Soc. C. E., has been published.§ The results are identical with those found by the use of Plate LXXXII. Therefore, no table for these discharges is presented in this paper.

* Continued from April, 1914, *Proceedings*.

† Author's closure.

‡ Blueprints (on cloth) of the original diagrams, and of diagrams more closely ruled, are made for field use.

§ "American Civil Engineers' Pocket Book", pp. 867-868.

Mr. Lyman.

TABLE 69.—DISCHARGES OVER SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS WITHOUT END CONTRACTIONS.

These Figures were Taken from Plate LXXX, and are Therefore Based on the Experiments Made by Bazin, Francis, Fteley and Stearns, and Also on Those Made in the Hydraulic Laboratories of Cornell University and the University of Utah. The Limits Included in the Diagram and the Table are Fully Covered by the Original Experiments.

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
2 3/8	0.200	0.315	0.314	0.313	0.312	0.311	0.310	0.309	
2 7/16	0.205	0.327	0.326	0.325	0.324	0.323	0.322	0.321	
2 1/2	0.210	0.340	0.337	0.336	0.335	0.334	0.333	0.332	
2 9/16	0.215	0.352	0.351	0.350	0.348	0.347	0.346	0.346	
2 5/8	0.220	0.365	0.363	0.360	0.359	0.357	0.356	0.355	
2 11/16	0.225	0.377	0.375	0.372	0.370	0.369	0.368	0.367	
2 3/4	0.230	0.392	0.388	0.385	0.383	0.382	0.381	0.380	
2 13/16	0.235	0.404	0.400	0.398	0.396	0.394	0.393	0.392	
2 7/8	0.240	0.420	0.415	0.412	0.408	0.406	0.405	0.404	
2 15/16	0.245	0.433	0.427	0.425	0.422	0.420	0.417	0.416	
3	0.250	0.446	0.442	0.438	0.435	0.434	0.432	0.430	
3 1/16	0.255	0.460	0.453	0.450	0.447	0.445	0.443	0.442	
3 1/8	0.260	0.475	0.468	0.465	0.460	0.458	0.456	0.455	
3 3/16	0.265	0.490	0.483	0.478	0.475	0.473	0.470	0.468	
3 1/4	0.270	0.503	0.497	0.493	0.488	0.486	0.484	0.483	
3 5/16	0.275	0.515	0.508	0.505	0.501	0.498	0.496	0.495	
3 3/8	0.280	0.530	0.524	0.518	0.514	0.510	0.507	0.506	
3 7/16	0.285	0.546	0.537	0.532	0.526	0.523	0.520	0.517	
3 1/2	0.290	0.560	0.552	0.547	0.544	0.540	0.535	0.533	
3 9/16	0.295	0.576	0.566	0.560	0.555	0.552	0.548	0.546	
3 5/8	0.300	0.595	0.584	0.576	0.570	0.566	0.563	0.560	
3 5/8	0.305	0.610	0.595	0.588	0.582	0.577	0.575	0.572	
3 11/16	0.310	0.625	0.612	0.605	0.598	0.595	0.590	0.586	
3 3/4	0.315	0.640	0.627	0.620	0.613	0.608	0.605	0.602	
3 13/16	3.020	0.655	0.645	0.636	0.630	0.625	0.620	0.617	
3 7/8	0.325	0.670	0.655	0.650	0.641	0.636	0.632	0.630	
3 15/16	0.330	0.690	0.672	0.665	0.656	0.652	0.647	0.645	
4	0.335	0.705	0.690	0.680	0.670	0.665	0.660	0.657	
4 1/16	0.340	0.720	0.705	0.697	0.688	0.683	0.675	0.673	
4 1/8	0.345	0.738	0.720	0.710	0.703	0.696	0.692	0.687	
4 3/16	0.350	0.755	0.735	0.726	0.717	0.712	0.705	0.702	
4 1/4	0.355	0.770	0.752	0.743	0.732	0.725	0.720	0.717	
4 5/16	0.360	0.790	0.772	0.760	0.750	0.745	0.737	0.733	
4 3/8	0.365	0.805	0.786	0.775	0.764	0.757	0.750	0.746	
4 7/16	0.370	0.824	0.802	0.792	0.780	0.775	0.766	0.762	
4 1/2	0.375	0.840	0.817	0.805	0.795	0.790	0.782	0.777	
4 9/16	0.380	0.860	0.836	0.825	0.813	0.805	0.798	0.795	
4 5/8	0.385	0.875	0.853	0.840	0.826	0.820	0.810	0.806	
4 11/16	0.390	0.896	0.870	0.857	0.845	0.837	0.830	0.825	
4 3/4	0.395	0.910	0.885	0.870	0.860	0.852	0.845	0.838	
4 13/16	0.400	0.930	0.905	0.893	0.875	0.870	0.860	0.855	0.850
4 7/8	0.405	0.950	0.922	0.910	0.895	0.885	0.875	0.870	0.860
4 15/16	0.410	0.970	0.940	0.925	0.910	0.903	0.895	0.885	0.876
5	0.415	0.990	0.956	0.943	0.925	0.917	0.908	0.903	0.895
5 1/16	0.420	1.005	0.975	0.958	0.943	0.935	0.924	0.917	0.910
5 1/8	0.425	1.020	0.995	0.977	0.963	0.952	0.942	0.935	0.926
5 1/8	0.430	1.045	1.010	0.996	0.980	0.970	0.957	0.952	0.945
5 3/16	0.435	1.065	1.030	1.010	0.996	0.986	0.975	0.970	0.960
5 1/4	0.440	1.083	1.045	1.026	1.010	1.000	0.992	0.985	0.976
5 5/16	0.445	1.100	1.063	1.045	1.026	1.015	1.005	1.000	0.994
5 3/8	0.450	1.120	1.080	1.060	1.040	1.030	1.015	1.010	1.030
5 7/16	0.455	1.140	1.100	1.080	1.057	1.047	0.035	1.023	1.016
5 1/2	0.460	1.164	1.125	1.105	1.085	1.074	1.056	1.050	1.043

TABLE 69.—(*Continued.*)

Mr. Lyman.

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
5 9/16	0.465	1.185	1.140	1.120	1.100	1.090	1.075	1.067	1.057
5 5/8	0.470	1.205	1.163	1.143	1.120	1.106	1.095	1.085	1.077
5 11/16	0.475	1.230	1.185	1.162	1.140	1.125	1.110	1.105	1.096
5 3/4	0.480	1.250	1.205	1.185	1.160	1.150	1.133	1.125	1.115
5 13/16	0.485	1.270	1.223	1.200	1.175	1.163	1.150	1.140	1.130
5 7/8	0.490	1.290	1.245	1.220	1.200	1.183	1.166	1.160	1.150
5 15/16	0.495	1.310	1.265	1.238	1.215	1.200	1.186	1.176	1.166
6	0.500	1.335	1.285	1.263	1.235	1.220	1.203	1.195	1.185
6 1/16	0.505	1.355	1.300	1.280	1.250	1.236	1.220	1.210	1.202
6 1/8	0.510	1.370	1.320	1.296	1.270	1.257	1.237	1.225	1.220
6 3/16	0.515	1.390	1.340	1.317	1.287	1.274	1.255	1.244	1.235
6 1/4	0.520	1.415	1.360	1.335	1.305	1.290	1.273	1.260	1.252
6 5/16	0.525	1.440	1.380	1.355	1.325	1.310	1.290	1.280	1.274
6 3/8	0.530	1.465	1.405	1.375	1.346	1.330	1.310	1.300	1.293
6 7/16	0.535	1.490	1.425	1.400	1.365	1.353	1.335	1.320	1.310
6 1/2	0.540	1.510	1.440	1.415	1.385	1.365	1.350	1.336	1.327
6 9/16	0.545	1.530	1.465	1.435	1.403	1.383	1.365	1.355	1.345
6 9/16	0.550	1.555	1.490	1.460	1.425	1.405	1.385	1.370	1.365
6 5/8	0.555	1.575	1.505	1.475	1.440	1.420	1.400	1.390	1.380
6 11/16	0.560	1.595	1.525	1.495	1.460	1.435	1.415	1.405	1.395
6 3/4	0.565	1.616	1.545	1.515	1.475	1.455	1.435	1.420	1.410
6 13/16	0.570	1.640	1.570	1.535	1.500	1.475	1.455	1.440	1.430
6 7/8	0.575	1.665	1.590	1.555	1.517	1.500	1.475	1.460	1.450
6 15/16	0.580	1.686	1.610	1.576	1.537	1.517	1.495	1.480	1.470
7	0.585	1.713	1.635	1.605	1.565	1.540	1.520	1.505	1.495
7 1/16	0.590	1.740	1.670	1.630	1.590	1.570	1.545	1.530	1.523
7 1/8	0.595	1.760	1.685	1.650	1.605	1.585	1.560	1.543	1.535
7 3/16	0.600	1.790	1.700	1.675	1.625	1.605	1.580	1.565	1.555
7 1/4	0.605	1.805	1.730	1.695	1.655	1.627	1.605	1.590	1.580
7 5/16	0.610	1.830	1.750	1.715	1.675	1.650	1.625	1.610	1.600
7 3/8	0.615	1.855	1.775	1.735	1.695	1.675	1.650	1.630	1.620
7 7/16	0.620	1.880	1.795	1.760	1.710	1.690	1.670	1.650	1.640
7 1/2	0.625	1.905	1.815	1.780	1.730	1.705	1.685	1.670	1.665
7 9/16	0.630	1.930	1.845	1.805	1.760	1.730	1.705	1.694	1.687
7 5/8	0.635	1.955	1.875	1.835	1.785	1.760	1.725	1.710	1.700
7 11/16	0.640	1.980	1.900	1.860	1.815	1.790	1.760	1.740	1.730
7 3/4	0.645	2.010	1.915	1.870	1.820	1.800	1.770	1.750	1.740
7 13/16	0.650	2.035	1.930	1.890	1.840	1.810	1.780	1.760	1.750
7 7/8	0.655	2.060	1.960	1.915	1.860	1.830	1.805	1.785	1.775
7 15/16	0.660	2.085	1.985	1.945	1.890	1.865	1.830	1.815	1.805
8	0.665	2.110	2.005	1.965	1.910	1.880	1.850	1.830	1.820
8 1/16	0.670	2.135	2.025	1.980	1.930	1.900	1.870	1.850	1.840
8 1/16	0.675	2.160	2.055	2.000	1.945	1.910	1.880	1.860	1.850
8 1/8	0.680	2.185	2.075	2.030	1.980	1.945	1.910	1.895	1.885
8 3/16	0.685	2.210	2.095	2.050	1.990	1.960	1.925	1.905	1.895
8 1/4	0.690	2.240	2.125	2.075	2.025	1.990	1.960	1.935	1.925
8 5/16	0.695	2.260	2.150	2.095	2.040	2.005	1.970	1.945	1.930
8 3/8	0.700	2.295	2.180	2.130	2.070	2.030	1.995	1.975	1.965
8 7/16	0.705	2.325	2.200	2.155	2.100	2.065	2.025	2.000	1.985
8 1/2	0.710	2.350	2.220	2.170	2.115	2.085	2.040	2.020	2.005
8 9/16	0.715	2.380	2.250	2.195	2.140	2.105	2.060	2.035	2.025
8 5/8	0.720	2.410	2.275	2.220	2.160	2.125	2.085	2.060	2.045
8 11/16	0.725	2.435	2.300	2.245	2.180	2.155	2.115	2.090	2.080
8 3/4	0.730	2.465	2.325	2.270	2.200	2.175	2.135	2.110	2.095
8 13/16	0.735	2.490	2.350	2.295	2.230	2.190	2.150	2.130	2.120
8 7/8	0.740	2.520	2.375	2.320	2.250	2.210	2.170	2.140	2.130
8 15/16	0.745	2.550	2.405	2.340	2.275	2.235	2.200	2.170	2.160
9	0.750	2.585	2.430	2.375	2.300	2.260	2.225	2.190	2.180
9 1/16	0.755	2.605	2.455	2.400	2.325	2.285	2.245	2.220	2.200
9 1/8	0.760	2.640	2.480	2.415	2.340	2.300	2.270	2.240	2.230
9 3/16	0.765	2.670	2.510	2.440	2.370	2.320	2.300	2.265	2.255
9 1/4	0.770	2.700	2.540	2.470	2.400	2.350	2.320	2.285	2.275
9 5/16	0.775	2.730	2.560	2.500	2.420	2.375	2.330	2.310	2.300
9 3/8	0.780	2.760	2.590	2.515	2.440	2.400	2.345	2.330	2.325
9 7/16	0.785	2.790	2.610	2.550	2.460	2.415	2.365	2.345	2.335
9 1/2	0.790	2.820	2.630	2.570	2.480	2.430	2.380	2.360	2.350
9 9/16	0.795	2.850	2.660	2.595	2.510	2.460	2.410	2.380	2.365

Mr. Lyman.

TABLE 69.—(*Continued.*)

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
9 1/16	0.800	2.890	2.700	2.625	2.550	2.500	2.440	2.410	2.400
9 1/8	0.805	2.910	2.730	2.660	2.575	2.520	2.465	2.425	2.410
9 1/16	0.810	2.940	2.755	2.680	2.595	2.545	2.485	2.445	2.425
9 1/4	0.815	2.975	2.780	2.700	2.610	2.565	2.505	2.460	2.440
9 5/16	0.820	3.010	2.810	2.735	2.640	2.590	2.530	2.500	2.480
9 3/8	0.825	3.045	2.840	2.770	2.670	2.610	2.560	2.530	2.510
9 5/16	0.830	3.070	2.870	2.790	2.700	2.640	2.580	2.550	2.535
10	0.835	3.100	2.905	2.830	2.730	2.675	2.610	2.580	2.565
10 1/16	0.840	3.130	2.930	2.840	2.760	2.695	2.630	2.600	2.590
10 1/8	0.845	3.160	2.950	2.880	2.785	2.730	2.650	2.615	2.605
10 3/16	0.850	3.190	2.990	2.910	2.800	2.750	2.680	2.650	2.630
10 1/4	0.855	3.230	3.015	2.930	2.840	2.780	2.710	2.670	2.650
10 5/16	0.860	3.260	3.040	2.960	2.860	2.800	2.735	2.700	2.680
10 3/8	0.865	3.290	3.070	2.980	2.880	2.815	2.750	2.715	2.695
10 7/16	0.870	3.320	3.100	3.010	2.910	2.840	2.780	2.740	2.720
10 1/2	0.875	3.350	3.120	3.035	2.930	2.870	2.795	2.765	2.750
10 9/16	0.880	3.395	3.160	3.070	2.965	2.900	2.820	2.790	2.780
10 5/8	0.885	3.415	3.180	3.090	2.980	2.920	2.840	2.810	2.790
10 11/16	0.890	3.445	3.200	3.120	3.010	2.940	2.860	2.825	2.820
10 3/4	0.895	3.480	3.235	3.150	3.040	2.970	2.895	2.860	2.845
10 13/16	0.900	3.520	3.270	3.180	3.070	3.000	2.920	2.890	2.870
10 7/8	0.905	3.550	3.300	3.210	3.100	3.035	2.940	2.910	2.890
10 15/16	0.910	3.580	3.330	3.235	3.120	3.055	2.970	2.930	2.910
11	0.915	3.620	3.360	3.260	3.155	3.085	3.000	2.955	2.935
11 1/16	0.920	3.655	3.390	3.290	3.180	3.110	3.030	2.980	2.960
11 1/8	0.925	3.690	3.420	3.325	3.210	3.140	3.055	3.010	2.990
11 3/16	0.930	3.720	3.445	3.350	3.230	3.160	3.075	3.030	3.010
11 1/4	0.935	3.760	3.480	3.380	3.250	3.180	3.100	3.060	3.040
11 5/16	0.940	3.800	3.510	3.405	3.290	3.210	3.130	3.080	3.060
11 3/8	0.945	3.830	3.540	3.430	3.315	3.240	3.150	3.110	3.090
11 7/16	0.950	3.870	3.580	3.470	3.350	3.260	3.180	3.140	3.120
11 1/2	0.955	3.900	3.610	3.500	3.380	3.295	3.200	3.165	3.140
11 9/16	0.960	3.940	3.640	3.540	3.400	3.325	3.235	3.190	3.170
11 5/8	0.965	3.980	3.680	3.570	3.430	3.355	3.260	3.210	3.190
11 11/16	0.970	4.010	3.700	3.590	3.450	3.370	3.275	3.235	3.200
11 3/4	0.975	4.040	3.740	3.625	3.490	3.405	3.310	3.270	3.250
11 13/16	0.980	4.080	3.770	3.650	3.520	3.430	3.330	3.290	3.270
11 7/8	0.985	4.120	3.800	3.690	3.555	3.460	3.365	3.320	3.300
11 15/16	0.990	4.150	3.830	3.710	3.580	3.480	3.380	3.340	3.320
12	0.995	4.180	3.850	3.730	3.590	3.510	3.400	3.360	3.330
12 1/8	1.000	4.230	3.900	3.780	3.640	3.555	3.440	3.400	3.375
12 1/4	1.010	4.300	3.970	3.840	3.710	3.600	3.500	3.450	3.420
12 3/8	1.020	4.380	4.030	3.900	3.760	3.670	3.560	3.500	3.480
12 1/2	1.030	4.450	4.100	3.970	3.820	3.720	3.600	3.560	3.540
12 5/8	1.040	4.520	4.170	4.040	3.880	3.780	3.670	3.620	3.590
12 3/4	1.050	4.610	4.240	4.120	3.950	3.850	3.730	3.670	3.650
12 11/16	1.060	4.800	4.320	4.180	4.020	3.910	3.790	3.740	3.710
12 13/16	1.070	4.760	4.370	4.220	4.070	3.960	3.830	3.770	3.750
12 15/16	1.080	4.820	4.430	4.280	4.130	4.010	3.890	3.820	3.800
13 1/16	1.090	4.900	4.480	4.340	4.180	4.060	3.930	3.870	3.840
13 3/16	1.100	4.980	4.570	4.420	4.240	4.140	3.990	3.940	3.910
13 5/16	1.110	5.060	4.640	4.480	4.320	4.190	4.060	4.000	3.960
13 7/16	1.120	5.150	4.710	4.560	4.370	4.240	4.120	4.050	4.010
13 9/16	1.130	5.220	4.780	4.610	4.420	4.300	4.170	4.100	4.070
13 11/16	1.140	5.300	4.840	4.670	4.480	4.360	4.210	4.160	4.130
13 13/16	1.150	5.380	4.910	4.740	4.560	4.420	4.270	4.210	4.180
13 15/16	1.160	5.450	4.980	4.800	4.610	4.480	4.330	4.260	4.220
14 1/16	1.170	5.510	5.050	4.870	4.670	4.540	4.380	4.320	4.280
14 1/8	1.180	5.600	5.130	4.950	4.740	4.610	4.440	4.380	4.340
14 1/4	1.190	5.680	5.200	5.000	4.800	4.660	4.500	4.420	4.400
14 3/8	1.200	5.780	5.250	5.075	4.870	4.720	4.560	4.480	4.440
14 1/2	1.210	5.860	5.340	5.150	4.940	4.780	4.610	4.540	4.500
14 5/8	1.220	5.940	5.420	5.250	5.000	4.860	4.680	4.610	4.590
14 3/4	1.230	6.000	5.460	5.270	5.050	4.910	4.720	4.640	4.610
14 7/8	1.240	6.100	5.550	5.360	5.150	4.980	4.800	4.720	4.680
15	1.250	6.200	5.620	5.430	5.220	5.050	4.860	4.780	4.740
15 1/8	1.260	6.275	5.675	5.500	5.275	5.100	4.910	4.830	4.800

TABLE 69.—(*Continued.*)

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Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
15¼	1.270		5.750	5.560	5.325	5.180	4.970	4.890	4.850
15⅜	1.280		5.820	5.620	5.380	5.225	5.000	4.940	4.900
15½	1.290		5.900	5.680	5.450	5.275	5.075	5.000	4.960
15⅝	1.300		5.975	5.775	5.525	5.350	5.150	5.050	5.020
15 11/16	1.310		6.060	5.850	5.600	5.425	5.225	5.130	5.080
15 12/16	1.320		6.150	5.920	5.675	5.500	5.275	5.200	5.150
15 13/16	1.330		6.200	6.000	5.730	5.550	5.350	5.250	5.220
16 1/16	1.340		6.300	6.050	5.800	5.620	5.400	5.320	5.260
16 2/16	1.350		6.375	6.130	5.875	5.675	5.460	5.370	5.320
16 3/16	1.360		6.450	6.200	5.940	5.750	5.520	5.430	5.380
16 4/16	1.370		6.505	6.300	6.000	5.820	5.580	5.500	5.450
16 5/16	1.380		6.625	6.375	6.080	5.900	5.650	5.560	5.525
16 11/16	1.390		6.700	6.450	6.150	5.960	5.725	5.625	5.575
16 12/16	1.400		6.780	6.530	6.230	6.040	5.770	5.675	5.640
16 13/16	1.410		6.860	6.620	6.320	6.100	5.850	5.760	5.700
17 1/16	1.420		6.950	6.675	6.375	6.150	5.920	5.820	5.760
17⅛	1.430		7.000	6.750	6.450	6.220	5.975	5.875	5.825
17¼	1.440		7.075	6.820	6.520	6.300	6.080	5.980	5.880
17⅜	1.450		7.150	6.900	6.600	6.360	6.100	6.000	5.950
17½	1.460		7.250	6.975	6.660	6.430	6.150	6.050	6.000
17⅝	1.470		7.330	7.050	6.740	6.500	6.220	6.120	6.060
17¾	1.480		7.400	7.130	6.800	6.508	6.300	6.175	6.125
17⅞	1.490		7.480	7.200	6.850	6.640	6.330	6.230	6.160
18	1.500		7.600	7.300	6.950	6.720	6.420	6.300	6.250
18⅛	1.510		7.660	7.360	7.020	6.775	6.500	6.360	6.300
18¼	1.520		7.750	7.450	7.100	6.850	6.550	6.450	6.360
18⅜	1.530		7.825	7.520	7.160	6.930	6.640	6.520	6.460
18½	1.540		7.900	7.600	7.230	7.000	6.680	6.575	6.500
18⅝	1.550		7.980	7.660	7.300	7.040	6.740	6.625	6.560
18 11/16	1.560		8.075	7.730	7.400	7.120	6.800	6.700	6.630
18 12/16	1.570		8.150	7.820	7.450	7.180	6.860	6.740	6.680
18 13/16	1.580		8.250	7.900	7.525	7.250	6.940	6.800	6.750
19 1/16	1.590		8.300	7.960	7.560	7.300	6.975	6.850	6.780

Table Giving Discharges for Weirs with Broad Crests.—Discharges over broad-crested weirs are given in Table 70. The matter it contains was prepared from Plate LXXXIII.

Suggestive Details for Weirs Recommended.—Suggestive details to be used in constructing weirs without end contractions are presented in Figs. 50, 51, and 52. It is recommended that the angle iron proposed for the crest have its end securely embedded in the walls of the channel; also that flush-gates, as shown in Figs. 50 and 51, be constructed so that the bottoms of the openings they cover will be on a level with the top of the floor of the channel of approach. If these flush-gates are of thin material, they can perhaps be located on the up-stream side of the bulkhead so that they will not affect materially the accuracy of the measurements. If supplied with lugs or iron loops, they may be pulled up and pushed down at pleasure.

Flush-Gates for Small Weirs.—For smaller weirs, the writer has used flush-gates of 1-in. lumber, with sheets of rubber gasket attached to them in such a way that they form a sort of flap or "check-valve". The pressure of the water on only one side of the gasket makes it fit close against the bottom of the channel and also around the three

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sides of the opening. A hole is cut in the rubber, and the edges of the material around this hole are fastened to the down-stream side of the gate so as to make a water-tight joint. The greater pressure thus brought to bear on the up-stream side of the gate brings it snugly against its seat, thus making it possible for the gasket to resist the water pressure.

Quantity of Water Each Weir Will Measure, and Quantity of Concrete Required to Construct Each Weir.—Suggestive dimensions of weirs to be used for measuring definite quantities of water and also the number of yards of concrete required to build these structures are given in Table 71.

The horizontal notch shown just below the crest of the weir and extending down stream to the end of the wall is constructed to allow the air free access to the under side of the falling sheet. Conditions are often such that a hole may be made through the wall of the channel just down stream from and below the crest of the weir, as at *A*, Fig. 51. This feature must not be overlooked when accurate measurements are desired.

Comparison of Results Given by the Diagrams and those Computed by the Francis Formula.—Mr. Moritz presents two ingenious and very convenient diagrams for finding the discharge over two weirs by the Francis formula. The scale of his diagram for suppressed weirs, however, is small. In order, therefore, to compare its results with those given by the diagrams presented in this paper, the quantities used have been computed by the Francis formula. The results of this comparison are given in Table 72.

The figures above the heavy line are the results obtained under the conditions specified by Mr. Francis himself, *viz.*, that with a "depth of channel leading to the weir * * * as small as three times that on the weir" the Francis formula "agrees with experiment within less than one per cent.", therefore "if the canal leading to the weir has a suitable depth, it will be requisite only when great precision is required * * * to go through the troublesome calculation of correcting the depth on the weir".*

Table 72 shows that, even when the conditions specified by Mr. Francis himself are satisfied (conditions which all experienced engineers attempt to follow), differences occur, varying from 1.3 to 7.0 per cent. Plate LXXXI shows the degree of accuracy with which the diagrams and tables herein presented fit the great mass of experimental data shown on Plates LXXXVI*a* and LXXXVI*b*. It should be noted, however, that the Francis data shown on Fig. 8, Plate LXXXVI*a*, on which the Francis formula is based, are but an exceedingly small fraction of the accepted data. If it be argued that these diagrams do not give results which are absolutely correct, it cannot be denied

* "Lowell Hydraulic Experiments", pp. 134 and 135.

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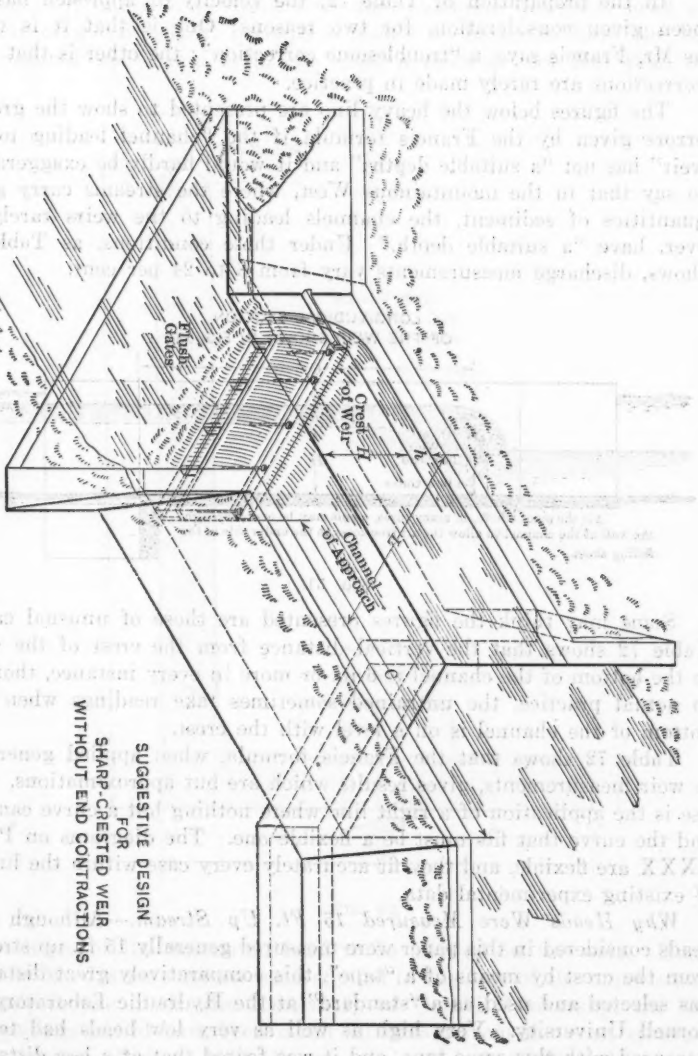

FIG. 50.

SUGGESTIVE DESIGN FOR SHARP-CRESTED WEIR WITHOUT END CONTRACTIONS

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that these results indicate positively that one right line or formula of the Francis form, namely, $Q = mh^n$ cannot be made to fit accurately these actual experimental results.

In the preparation of Table 72, the velocity of approach has not been given consideration, for two reasons: One is that it is truly, as Mr. Francis says, a "troublesome correction"; the other is that such corrections are rarely made in practice.

The figures below the heavy line are presented to show the greater errors given by the Francis formula if the "channel leading to the weir" has not "a suitable depth;" and it would hardly be exaggerating to say that in the mountainous West, where the streams carry great quantities of sediment, the channels leading to the weirs rarely, if ever, have "a suitable depth." Under these conditions, as Table 72 shows, discharge measurements vary from 8 to 24 per cent.

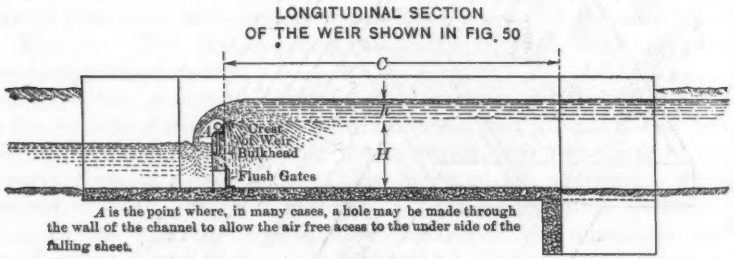

FIG. 51.

Some may think the figures presented are those of unusual cases. Table 72 shows that the vertical distance from the crest of the weir to the bottom of the channel is 6 in. or more in every instance, though, in actual practice, the untrained sometimes take readings when the bottom of the channel is on a level with the crest.

Table 72 shows that the Francis formula, when applied generally to weir measurements, gives results which are but approximations. Its use is the application of a right line where nothing but a curve can fit, and the curve that fits must be a flexible one. The diagrams on Plate LXXX are flexible, and they fit accurately every case within the limits of existing experimental data.

Why Heads Were Measured 15 Ft. Up Stream.—Although the heads considered in this paper were measured generally 15 ft. up stream from the crest by means of a "tape", this comparatively great distance was selected and used as a "standard" at the Hydraulic Laboratory of Cornell University. Very high as well as very low heads had to be measured with this same tape, and it was feared that at a less distance the "curved surface over the crest" might affect the readings for high heads. The cross-section of the channel of approach was so great, and

the floor in it so nearly horizontal, that the results thus obtained are perhaps the same as if readings had been taken as Mr. Hazen suggests, at a distance up stream "equal to 2.5 times the height of the weir." However, unless some limit fixes the maximum allowable head on a weir of given height, perhaps it would be better to have the heads measured up stream from the crest a distance not less than some

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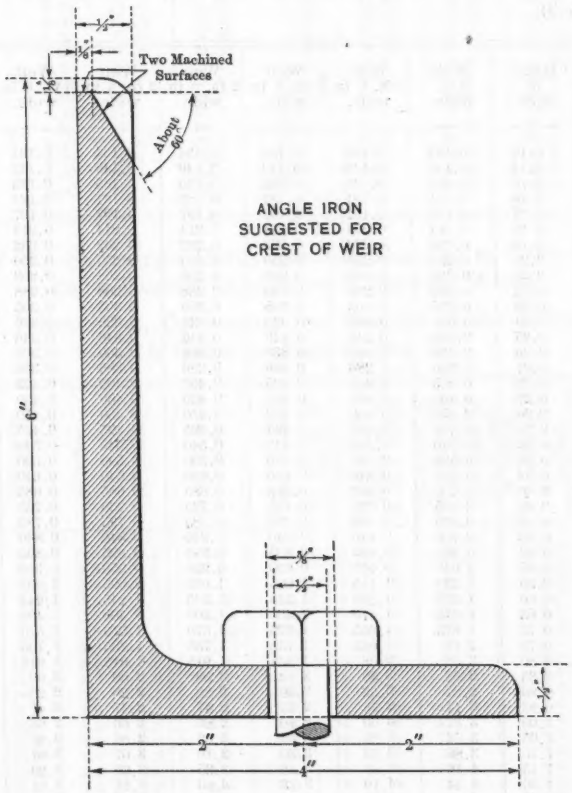

FIG. 52.

multiple of the maximum head to be allowed on the weir. Had the rule Mr. Hazen suggests been applied in the case of the upper standard weir in the Hydraulic Laboratory of Cornell University, the "tape" readings would have been taken up stream from the crest of the weir a distance of about 28 ft., because the weir is 11.25 ft. high.

Weirs Having Dimensions Proportional.—Concerning one other of the many excellent matters presented in the discussion of Mr. Hazen,

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TABLE 70.—DISCHARGE, IN CUBIC FEET PER SECOND PER FOOT OF LENGTH, OVER BROAD-CRESTED WEIRS WITHOUT END CONTRACTIONS.

These Quantities Were Read from Plate LXXXIII, Which Is Based on Experiments Made on Weirs 11.25 Ft. High in the Hydraulic Laboratory of Cornell University. For the Discharge over Weirs of Heights Other than 11.25 Ft., Apply the Coefficients Given in Table 0.

Head, in inches.	Head, in feet.	Weir 6 in. wide.	Weir 1 ft. 0 in. wide.	Weir 1 ft. 6 in. wide.	Weir 2 ft. 0 in. wide.	Weir 2 ft. 6 in. wide.	Weir 3 ft. 0 in. wide.	Weir 3 ft. 6 in. wide.
1 1/16	0.13	0.134	0.134	0.134	0.134	0.134	0.134	0.134
1 1/16	0.14	0.149	0.149	0.149	0.149	0.149	0.149	0.149
1 1/16	0.15	0.165	0.165	0.165	0.165	0.165	0.165	0.165
1 5/16	0.16	0.181	0.181	0.181	0.181	0.181	0.181	0.181
2 1/16	0.17	0.197	0.197	0.197	0.197	0.197	0.197	0.197
2 3/16	0.18	0.214	0.214	0.214	0.214	0.214	0.214	0.214
2 1/4	0.19	0.232	0.232	0.232	0.232	0.232	0.232	0.232
2 3/8	0.20	0.250	0.250	0.250	0.250	0.250	0.250	0.250
2 1/2	0.21	0.268	0.268	0.268	0.268	0.268	0.268	0.268
2 5/8	0.22	0.288	0.288	0.288	0.288	0.288	0.288	0.288
2 3/4	0.23	0.305	0.305	0.305	0.305	0.305	0.305	0.305
2 7/8	0.24	0.326	0.326	0.326	0.326	0.326	0.326	0.326
3	0.25	0.346	0.346	0.346	0.346	0.346	0.346	0.346
3 1/8	0.26	0.366	0.366	0.366	0.366	0.366	0.366	0.366
3 1/4	0.27	0.386	0.386	0.386	0.386	0.386	0.386	0.386
3 3/8	0.28	0.405	0.405	0.405	0.405	0.405	0.405	0.405
3 1/2	0.29	0.430	0.430	0.430	0.430	0.430	0.430	0.430
3 5/8	0.30	0.450	0.450	0.450	0.450	0.450	0.450	0.450
3 13/16	0.32	0.495	0.495	0.495	0.495	0.495	0.495	0.495
4 1/16	0.34	0.540	0.540	0.540	0.540	0.540	0.540	0.540
4 5/16	0.36	0.590	0.590	0.590	0.590	0.590	0.590	0.590
4 9/16	0.38	0.645	0.640	0.640	0.640	0.640	0.640	0.640
4 13/16	0.40	0.705	0.685	0.685	0.685	0.685	0.685	0.685
5 1/16	0.42	0.765	0.735	0.735	0.735	0.735	0.735	0.735
5 1/4	0.44	0.830	0.785	0.785	0.785	0.785	0.785	0.785
5 1/2	0.46	0.900	0.840	0.840	0.840	0.840	0.840	0.840
5 3/4	0.48	0.970	0.895	0.895	0.895	0.895	0.895	0.895
6	0.50	1.040	0.950	0.950	0.950	0.950	0.950	0.950
6 5/8	0.55	1.230	1.115	1.095	1.095	1.095	1.095	1.095
7 3/16	0.60	1.425	1.285	1.245	1.245	1.245	1.245	1.245
7 13/16	0.65	1.645	1.470	1.400	1.400	1.400	1.400	1.400
8 3/8	0.70	1.875	1.665	1.570	1.570	1.570	1.570	1.570
9	0.75	2.11	1.865	1.735	1.735	1.735	1.735	1.735
9 5/8	0.80	2.36	2.08	1.935	1.915	1.915	1.915	1.915
10 5/16	0.85	2.61	2.30	2.14	2.09	2.09	2.09	2.09
10 13/16	0.90	2.87	2.52	2.36	2.28	2.28	2.28	2.28
11 3/8	0.95	3.11	2.76	2.57	2.47	2.47	2.47	2.47
12	1.00	3.37	3.00	2.80	2.66	2.66	2.66	2.66
12 5/8	1.05	3.62	3.26	3.04	2.86	2.86	2.86	2.86
13 3/16	1.10	3.88	3.53	3.30	3.10	3.07	3.07	3.07
13 13/16	1.15	4.16	3.80	3.56	3.35	3.29	3.29	3.29
14 3/8	1.20	4.44	4.10	3.83	3.60	3.51	3.51	3.51
15	1.25	4.70	4.38	4.10	3.85	3.72	3.72	3.72
15 5/8	1.30	5.00	4.68	4.38	4.12	3.96	3.95	3.95
16 13/16	1.40	5.58	5.28	4.94	4.66	4.48	4.40	4.40
18	1.50	6.16	5.92	5.56	5.24	5.02	4.88	4.88
19 3/16	1.60	6.76	6.54	6.16	5.82	5.58	5.42	5.36
20 3/8	1.70	7.40	7.24	6.82	6.42	6.18	6.00	5.86
21 5/8	1.80	8.04	7.94	7.50	7.08	6.78	6.58	6.42
22 13/16	1.90	8.76	8.74	8.22	7.76	7.44	7.22	7.02
24	2.00	9.42	9.42	8.96	8.44	8.08	7.84	7.64
25 5/16	2.10	10.20	10.20	9.70	9.16	8.78	8.50	8.26
26 5/8	2.20	10.95	10.95	10.50	9.90	9.50	9.18	8.94
27 5/8	2.30	11.70	11.70	11.30	10.65	10.20	9.88	9.60
28 13/16	2.40	12.50	12.50	12.10	11.45	10.95	10.60	10.30
30	2.50	13.30	13.30	12.95	12.20	11.70	11.35	11.00

TABLE 70.—(*Continued.*)

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Head, in inches.	Head, in feet.	Weir 6 in. wide.	Weir 1 ft. 0 in. wide.	Weir 1 ft. 6 in. wide.	Weir 2 ft. 0 in. wide.	Weir 2 ft. 6 in. wide.	Weir 3 ft. 0 in. wide.	Weir 3 ft. 6 in. wide.
31 3/16	2.60	14.15	14.15	13.85	13.10	12.55	12.10	11.80
32 2/8	2.70	15.00	15.00	14.75	13.95	13.35	12.90	12.55
33 5/8	2.80	15.90	15.90	15.70	14.80	14.15	13.70	13.30
34 13/16	2.90	16.75	16.75	16.60	15.70	15.00	14.50	14.05
36	3.00	17.65	17.65	17.60	16.60	15.90	15.35	14.90
38 4/8	3.20	19.50	19.50	19.50	18.50	17.70	17.05	16.55
40 13/16	3.40	21.5	21.5	21.5	20.5	19.60	18.90	18.25
43 1/16	3.60	23.5	23.5	23.5	22.6	21.6	20.8	20.1
45 5/8	3.80	25.4	25.4	25.4	24.7	23.6	22.7	22.0
48	4.00	27.6	27.6	27.6	26.9	25.8	24.8	24.0
50 4/8	4.20	29.7	29.7	29.7	29.2	27.9	26.8	25.8
52 13/16	4.40	31.8	31.8	31.8	31.5	30.0	28.9	27.8
55 1/16	4.60	34.2	34.2	34.2	34.0	32.4	31.1	30.0
57 5/8	4.80	36.6	36.6	36.6	36.5	34.9	33.5	32.2
60	5.00	38.8	38.8	38.8	38.8	37.3	35.8	34.5

TABLE 71.—SUGGESTIVE DIMENSIONS FOR WEIRS TO BE USED FOR MEASURING VARIOUS QUANTITIES OF FLOWING WATER.

Further Information, Concerning the Meaning of the Notation Used, is Given on Figs. 50 and 51.

Minimum, Q, in cubic feet.	Maximum, Q, in cubic feet.	h, in feet.	H, in feet.	Height of wall, in feet.	L, length of weir, in feet.	C, length of channel, in feet.	Thickness of floor, in inches.	Thickness of bottom of wall, in inches.	Thickness of top of wall, in inches.	Length of wing wall, in feet.	Depth of cut-off wall, in feet.	Concrete, in cubic yards.
0.05	0.25	0.5	0.5	1.0	0.2	2	6	8	6	2.0	1.0	0.5
0.15	0.60	0.5	0.5	1.0	0.5	4	6	8	6	2.0	1.0	0.7
0.30	1.30	0.5	0.5	1.0	1.0	6	6	8	6	2.0	1.0	0.9
0.45	1.90	0.5	0.75	1.25	1.5	8	6	8	6	2.0	1.0	1.4
0.60	2.50	0.5	0.75	1.25	2.0	10	6	8	6	2.5	1.0	2.2
0.75	3.2	0.5	0.75	1.25	2.5	10	6	8	6	2.5	1.0	2.3
1.0	3.7	0.5	1.0	1.5	3.0	10	6	8	6	2.5	1.0	2.4
1.5	11.0	1.0	1.0	2.0	3.5	12	6	8	6	3.0	1.0	3.5
2.0	15.0	1.0	1.0	2.0	4.0	12	6	8	6	3.0	1.0	3.7
2.5	18.0	1.0	1.5	2.5	5.0	12	6	8	6	3.0	1.0	4.0
3.0	21.0	1.0	1.5	2.5	6.0	14	8	12	6	3.0	1.5	5.7
3.5	25.0	1.0	1.5	2.5	7.0	14	8	12	6	4.0	1.5	6.2
4.0	28.0	1.0	2.0	3.0	8.0	14	8	12	6	5.0	1.5	10.0
4.5	32.0	1.0	2.0	3.0	9.0	16	8	12	8	5.0	1.5	11.4
5.0	65.0	1.5	2.0	3.5	10.0	16	8	12	8	8.0	2.0	18.0
6.0	71.0	1.5	2.0	3.5	11.0	16	8	12	8	8.0	2.0	18.7
7.0	78.0	1.5	2.0	3.5	12.0	16	12	15	8	8.0	2.0	24.5
8.0	84.0	1.5	2.0	3.5	13.0	16	12	15	8	8.0	2.0	25.2
9.0	91.0	1.5	3.0	4.5	14.0	18	12	15	8	9.0	2.5	34.6
10.0	97.0	1.5	3.0	4.5	15.0	18	12	18	12	10.0	3.0	44.6
11.0	104.0	1.5	3.0	4.5	16.0	18	12	18	12	10.0	3.0	47.5
12.0	170.0	2.0	3.0	5.0	17.0	18	12	18	12	10.0	3.0	48.7
13.0	180.0	2.0	3.0	5.0	18.0	18	12	18	12	10.0	3.0	49.9
14.0	200.0	2.0	3.0	5.0	20.0	20	12	18	12	10.0	3.0	55.5
15.0	215.0	2.0	4.0	6.0	22.0	20	12	24	12	12.0	4.0	78.0
16.0	230.0	2.0	4.0	6.0	24.0	20	12	24	12	12.0	4.0	80.8
17.0	250.0	2.0	4.0	6.0	26.0	20	12	24	12	12.0	4.0	83.6
18.0	270.0	2.0	4.0	6.0	28.0	20	12	24	12	12.0	4.0	86.4
19.0	290.0	2.0	4.0	6.0	30.0	20	12	24	12	12.0	4.0	89.2
20.0	310.0	2.0	4.0	6.0	32.0	20	12	24	12	12.0	4.0	92.2

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viz., "that weirs which, in all respects, are proportional to each other will have the same coefficients", the writer wishes to say that for years he has attempted to get into clear form this same idea. Now that this principle has been clearly stated, he will attempt to demonstrate in the Hydraulic Laboratory of the University of Utah whether or not it is correct.

TABLE 72.—PERCENTAGE OF DIFFERENCE BETWEEN QUANTITIES OF DISCHARGE OVER WEIRS OF VARIOUS HEIGHTS, AS GIVEN BY PLATE LXXX, AND AS COMPUTED BY THE FRANCIS FORMULA: $Q_F = 3.33\,H^{\frac{3}{2}}$.

Q_F = Discharge, in cubic feet per second.

H = Height of weir, in feet.

The quantities above the heavy line are within the limits for the formula specified by Mr. Francis.

Head, in feet.	Q_F	Quantity, as Read from Plate LXXX.							
		$H = 0.5$ ft.	$H = 0.75$ ft.	$H = 1.0$ ft.	$H = 1.5$ ft.	$H = 2.0$ ft.	$H = 3.0$ ft.	$H = 4.0$ ft.	$H = 6.0$ ft.
0.2	0.298	0.315 5.4%	0.314 5.1%	0.313 4.8%	0.312 4.5%	0.311 4.2%	0.310 3.9%	0.309 3.6%	
0.3	0.547	0.595 8.1%	0.584 6.3%	0.576 5.0%	0.570 4.0%	0.566 3.4%	0.563 2.8%	0.560 2.3%	
0.4	0.842	0.930 9.5%	0.905 7.0%	0.893 5.7%	0.875 3.8%	0.870 3.2%	0.860 2.1%	0.855 1.5%	0.850 0.9%
0.7	1.950	2.295 15.0%	2.180 10.5%	2.130 8.5%	2.070 5.8%	2.030 3.9%	1.995 2.3%	1.975 1.3%	1.965 0.8%
1.0	3.330	4.230 21.3%	3.900 14.6%	3.780 11.9%	3.640 8.5%	3.555 6.3%	3.440 3.2%	3.400 2.1%	3.375 1.6%
1.3	4.936	6.51 24.2%	5.975 17.4%	5.775 14.5%	5.525 10.7%	5.350 7.7%	5.150 4.2%	5.050 2.3%	5.020 1.7%
1.6	6.789	....	8.40 19.8%	8.07 16.5%	7.69 12.4%	7.41 9.2%	7.08 4.8%	6.94 3.0%	6.89 2.3%

The Weir Without End Contractions Gives the Most Accurate Results.—Mr. Moritz says the statement that the "sharp-crested weir without end contractions" gives "the most accurate results with the smallest amount of work * * * is open to serious question", as the use of such weirs "would be practically impossible on account of the large quantity of silt carried at certain seasons of the year" in the canals of some irrigation projects.

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Great Velocity of Approach and Little Sediment Deposited.—The first of the six reasons why these weirs are best, given near the beginning of this paper, was that, with this structure, the velocity of the water above the weir is greatest, and therefore its capacity for carrying silt over the crest is also greatest. The second of the six reasons is that, with this weir, the channel of approach must be made of concrete, lumber, or some other material, that will retain a definite rectangular form; it is easy, therefore, to determine whether or not the channel is clean, and it must be clean to give good results. Another reason may now be added: This weir is best because its crest may be fixed in position permanently, and flush-gates may be built under it. Through these gates silt may pass when measurements are not actually being taken. Thus is maintained a relatively great velocity in this comparatively small channel of approach, and the channel is kept clean.

The Diagrams and Tables Fit Great Variety of Weirs and Great Variation of Heads.—Again, as previously explained, the diagrams and tables submitted in this paper (Plate LXXX and Table 69) fit, and fit accurately, a great variety of weirs of this type, and do so for a comparatively large range of heads. The Francis diagram or formula for suppressed weirs, based on only 17 experiments, cannot be expected to fit, nor does it fit, the great variety of weirs and the large range of heads to which it has been and is being applied. The smallest head used by Mr. Francis was greater than 0.73 ft., and the largest less than 1.1 ft.; his experiments, therefore, cover a difference in head of but little more than 3 in. Moreover, all his experiments were made on only one weir with only one height.

Francis Formula Fits Within Very Narrow Limits.—The Francis experiments are shown on Fig. 8, Plate LXXXVI*a*. The results obtained by other experiments are presented on both sides of this figure. They indicate that if Francis had used higher and lower heads, he would have discovered that no one equation of the form he uses ($Q = mh^n$) can fit accurately a series of experiments on weirs of the type herein considered.

The Best Practical Measuring Device Available.—Mr. Moritz closes his argument in favor of the Cippoletti weir with these words:

"In the writer's opinion, there is no better practical device, at present available, than the Cippoletti weir for measuring the quantities encountered in the majority of irrigation laterals."

The velocity of the water approaching the Cippoletti weir is less than that of the water approaching the weir without end contractions. In order to give accurate results, the cross-section of the channel of approach for the former must be greater than that for the latter. The deposit of silt, therefore, will be greater above the Cippoletti weir than above the weir without end contractions. If, then, with the Cippoletti

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weir, as he explains, "one or two good cleanings of the pool during the season will generally suffice", less cleaning will be required for the weir without end contractions. Still more important is the fact that a conscientious water master may clean his canal so as to make a large pool, and a water master of the other sort may make but a small pool, or none at all. Table 72 shows what effect a change in the cross-section of the "pool" or channel of approach has on the discharge. In the case of the suppressed weir, the channel to be cleaned has a fixed cross-section, of concrete, timber, or other material, that will retain its definite rectangular section. With sluice-gates through the bulkhead under the crest of the weir, this channel can be cleaned with the aid of the water much more easily than can the "pool" of which Mr. Moritz speaks.

If the channel above the suppressed weir has 6 in. of silt in it, the discharge (by taking this fact into account) can be found from the diagrams and tables herewith submitted. This cannot be done with the Cippoletti weir.

The writer does not see wherein the use of the tables and diagrams recommended in this paper requires any greater care than does the use of the Cippoletti weir and the diagram Mr. Moritz submits. Neither does he concede that the Cippoletti weir is the simpler of the two, in the sense of its being easier to use; but he will concede that it may be installed at a little less cost.

Mr. Moritz does not give a reference to the experiments which he states show that the Cippoletti weir gives results "within an error of from 2 to 4% when the head is not greater than one-third of the crest length." However, as the Francis formula is used for computing the discharge over the Cippoletti weir, it is important to specify the distance of each contraction from the sides and from the bottom of the channel. Of course, the greater the cross-section of the channel of approach, the more nearly will conditions approach the ideal; and the nearer this ideal is approached, the greater will be the tendency to fill the channel with silt.

Accuracy of Results.—Mr. Moritz claims that the Cippoletti weir, under what he intimates are rather limited conditions, gives results "within an error of from 2 to 4%"; but he fails to give the quantity or value of the data on which he bases this assertion. He does explain, however, that "the quantities in these experiments were measured volumetrically", and intimates in the same sentence that there are errors in the tables and diagrams herein presented "due to the use of a calibrated measuring device * * * such as the 'standard weir' used in the Cornell Experiments," giving results which may be "accepted as accurate to within 2 or 3 per cent."

Evidently, Mr. Moritz has gone carefully over the matter presented in this paper, but he did not see for the moment that the "2 or 3 per

cent." applies to "Broad-Crested Weirs" (page 1523*), and not to "Sharp-Crested Weirs Without End Contractions" (page 1516*). It is for the latter weirs and for the diagrams and tables which accompany them that this paper makes its greatest claim. The results are based on the twenty "classic" series of experiments named on the left of Plate LXXXI. This plate indicates clearly with what degree of accuracy the diagrams and tables of this paper fit this unparalleled array of hydraulic data.† Mr. Lyman.

Discrepancies in Quantities for Weirs with Broad Crests.—To furnish the "additional light" for which Mr. Moritz asks concerning certain runs on broad-crested weirs, Fig. 53 has been made. The curves shown are drawn to fit the points based on the original experiments. These points are represented by large dots. The results reached by Mr. Williams‡ are shown by small crosses, and those obtained by Mr. Horton§ are represented by small dots within triangles.

The accuracy with which the results given by the diagrams on Plates LXXXV and XC fit those obtained by using the original data is shown by the circles on Fig. 53. This information and the original data on which Messrs. Williams and Horton also based their results are given in Table 73. From the quantities given, and their signs, in the column marked "Percentage of difference", it is seen that the results obtained by using Plates LXXXV and XC fit the original experiments almost perfectly. There are only five individual experiments which show a difference greater than 2%, and two of these would show still smaller differences but for the fact that slight errors in calculation were made in the original computations.

Accuracy with Which Diagrams Fit Original Data.—Attention is again drawn to Plates LXXXIX and XCI*a*. These plates show that the diagrams giving mean values furnish results which can be used with greater confidence than can the results of the individual runs. Other plates in this paper give the same information concerning the accuracy of the other diagrams.

Although the Suppressed Weir Is Recommended as the Best Weir, Other Measuring Devices Are Necessary.—It was not, as Mr. Johnston seems to have understood, the writer's intention that the weir be adopted by legislation as the only device for measuring water. The intention was to advocate that, by legislation, the weir without end contractions be adopted as the "standard weir". It can be used to obtain accurate results wherever any other type of weir can be used, and in many places and under many conditions where reliance can be placed on no other weir. If, as Mr. Robinson says, this type of weir

* *Proceedings*, Am. Soc. C. E., for September, 1913.

† Reference is given in the proper places in this paper to the original sources from which these data came.

‡ "American Civil Engineers Pocket Book," p. 869.

§ U. S. Geological Survey, Water Supply Paper No. 200, pp. 190 and 192.

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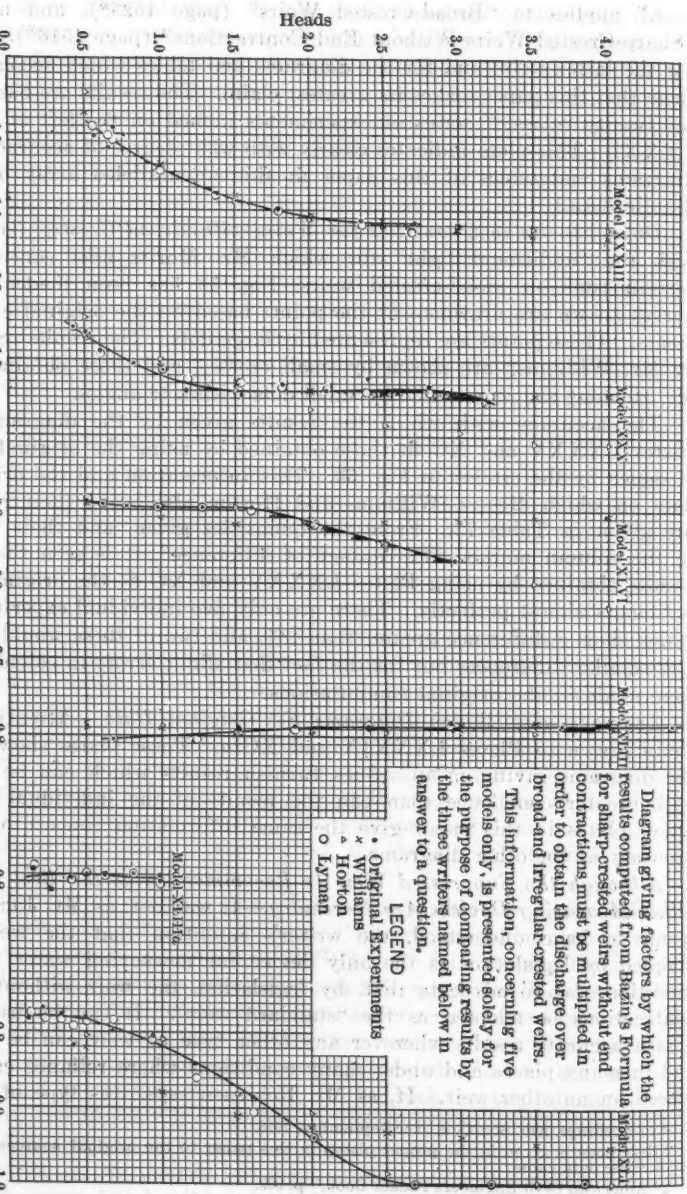

Diagram giving factors by which the results computed from Bazin's Formula Model XLII for sharp-crested weirs without end contractions must be multiplied in order to obtain the discharge over broad-and irregular-crested weirs.

This information, concerning five models only, is presented solely for the purpose of comparing results by the three writers named below in answer to a question.

Fig. 53.

were made standard by Federal Statute, then it would soon be used universally. By further actual experiment on this one weir, if the results based on the data at present available are not absolutely correct, data will be obtained from which more and more accurate results can be found. Mr. Lyman.

With this suggested standard weir, it is believed, gravel and silt can be disposed of, as already explained, without constructing the weir in a channel parallel to the main canal, as Mr. Johnston suggests.

Basic Theory on Which Diagrams Are Drawn.—As it was the intention to give the results of this investigation first and the theory afterward, it is not surprising that Mr. Johnston, in his discussion, states that the basic theory of the method used is not explained. He also says, after stating that the differences used cannot be shown with sufficient accuracy, when the rectangular axes are used, "it would seem to one who has not given extended study to the problem that Mr. Lyman continues to use rectangular axes."

On all the plates intended for giving discharges, it may be seen that the lines representing heads and those representing discharges are not at right angles to one another.

The beginning of the explanation of the graphic method used in the construction of these diagrams is on page 1534, headed "E.—Method of Determining the Proposed Formulas." Unfortunately, on page 1545 a reference to "Fig. 2, Plate LXXXVI*a* (right hand end)" should be to "Fig. A, Plate LXXXVI*b* (right hand end)". Further explanation of the theory on which the construction of these diagrams is based, and particularly that portion relating to the inclination of the axes is given beginning on page 1566 in the part entitled "I.—Method of Constructing the Final Diagrams."

Although Mr. Johnston's "basic theory" is certainly ingenious, and of intense interest from the mathematical point of view, it could perhaps not be applied so easily to the construction of the diagrams herein given as can the method, largely graphical, which was actually used in their construction.

The Results Fit Perfectly the Conditions Prevailing in Ordinary Irrigation Streams.—Mr. Winsor states, with respect to the experiments made on weirs without end contractions:

"Most of the observations thus far have been conducted under conditions quite foreign to those which are met in the operation of a canal system. The early experimenters used, in the main, larger streams than the ordinary irrigation stream requiring measurements, and the recent experiments were conducted under laboratory conditions which are seldom applicable to the ordinary irrigation stream."

The heads presented in this paper were measured with a "tape", or else have been reduced to equivalent "tape readings"; heads can be thus conveniently measured in any canal. Mr. Winsor did not suggest a simpler or more practical method. If the water in the canal system

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TABLE 73.—DIFFERENCES IN PERCENTAGE BETWEEN DISCHARGES OVER BROAD-CRESTED WEIRS AS ACTUALLY MEASURED AND AS READ FROM PLATES LXXXV AND XC; ALSO FACTORS BY WHICH THE RESULTS COMPUTED FROM BAZIN'S FORMULA FOR SHARP-CRESTED WEIRS WITHOUT END CONTRACTIONS MUST BE MULTIPLIED IN ORDER TO OBTAIN THE DISCHARGE OVER BROAD-CRESTED AND IRREGULAR-CRESTED WEIRS.

Q_m = Discharge, in Second-Feet, as Actually Measured.

Q_L = Discharge, in Second-Feet, as Read from Plates LXXXV and XC.

Q_B = Discharge, in Second-Feet, for Sharp-Crested Weirs, as Computed by Bazin's Formula.

$$M_m = \frac{Q_m}{Q_B}.$$

$$M_L = \frac{Q_L}{Q_B}.$$

MODEL XXXIII.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
2.660	16.272	16.45	−1.09	14.55	1.118	1.130
2.352	13.464	13.55	−0.64	12.07	1.116	1.123
1.784	8.747	8.76	−0.15	7.94	1.101	1.102
1.394	5.914	5.94	−0.44	5.475	1.079	1.085
0.992	3.439	3.460	−0.61	3.300	1.042	1.048
0.752	2.197	2.240	−1.96	2.192	1.001	1.022
0.679	1.866	1.896	−1.61	1.882	0.992	1.007
0.653	1.808	1.785	+1.27	1.775	1.018	1.004
0.553	1.397	1.375	+1.57	1.389	1.006	0.992

MODEL XXX.

3.187	20.152	20.00	+0.75	19.17	1.051	1.043
2.790	16.219	16.42	−1.24	15.65	1.037	1.049
2.384	12.631	12.88	−1.97	12.35	1.022	1.042
2.094	10.577	10.55	−0.26	10.15	1.041	1.039
1.793	8.282	8.30	−0.22	8.00	1.034	1.037
1.532	6.607	6.504	+1.56	6.317	1.045	1.030
1.174	4.399	4.350	+1.11	4.238	1.039	1.027
1.010	3.436	3.430	+0.18	3.390	1.012	1.011
0.802	2.409	2.416	−0.29	2.409	1.000	1.002
0.613	1.608	1.600	+0.48	1.621	0.992	0.997
0.508	1.195	1.200	−0.42	1.239	0.966	0.970
0.427	0.921	0.920	+0.11	0.960	0.959	0.958

MODEL XLVI.

2.965	14.901	14.85	+0.34	17.18	0.868	0.865
2.486	11.032	11.13	−0.89	13.13	0.841	0.848
2.030	7.895	7.95	−0.70	9.68	0.816	0.822
1.597	5.360	5.38	−0.37	6.711	0.798	0.803
1.232	3.628	3.624	+0.11	4.556	0.797	0.799
0.972	2.549	2.549	±0.00	3.201	0.797	0.798
0.784	1.856	1.857	−0.05	2.328	0.797	0.797
0.602	1.255	1.256	−0.08	1.578	0.796	0.796
0.503	0.967	0.963	+0.41	1.222	0.791	0.788

TABLE 73.—(*Continued.*)

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MODEL XLIII.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
4.432	25.011	24.94	+0.28	31.95	0.783	0.781
3.661	18.651	18.69	−0.21	23.78	0.785	0.786
2.935	13.200	13.27	−0.53	16.94	0.779	0.783
2.360	9.608	9.54	+0.71	12.15	0.791	0.784
1.890	6.841	6.87	−0.42	8.66	0.789	0.794
1.480	4.767	4.78	−0.06	6.012	0.793	0.796
1.206	3.520	3.550	−0.85	4.413	0.798	0.805
0.689	1.544	1.545	−0.07	1.924	0.803	0.804

MODEL XLIII*a*.

0.986	2.618	2.625	−0.27	3.270	0.800	0.803
0.981	2.568	2.605	−1.44	3.245	0.791	0.803
0.786	1.847	1.850	−0.16	2.337	0.791	0.793
0.621	1.309	1.310	−0.08	1.653	0.792	0.793
0.496	0.945	0.942	+0.32	1.196	0.790	0.788
0.392	0.671	0.674	−0.45	0.844	0.795	0.798
0.279	0.421	0.409	+2.85	0.520	0.810	0.787
0.161	0.184	0.185	−1.09	0.238	0.774	0.778

MODEL XLI.

3.841	25.531	25.47	+0.24	25.59	0.999	0.999
3.176	18.995	19.04	−0.24	19.06	0.998	0.999
2.674	14.624	14.36	+1.80	14.66	0.999	0.999
2.021	9.021	9.00	+0.23	9.61	0.938	0.937
1.601	5.936	6.08	−2.42	6.72	0.883	0.905
1.233	3.835	3.920	−2.21	4.562	0.841	0.860
0.941	2.476	2.485	−0.36	3.058	0.810	0.814
0.670	1.472	1.442	+2.04	1.845	0.798	0.782
0.488	0.911	0.917	−0.66	1.167	0.781	0.786
0.417	0.759	0.735	+3.16	0.926	0.819	0.794
0.330	0.520	0.526	−1.15	0.669	0.777	0.787
0.210	0.273	0.278	−1.83	0.355	0.769	0.783
0.122	0.130	0.128	+1.54	0.166	0.784	0.772

is measured with any degree of care, then, with the heads measured as above, the conditions under which these experiments were made appear to be identical with those "met in the operation of a canal system."

We may wait long for many series of experiments to be made with such care that they will deserve a place among those used in this paper. It will certainly be a much longer time before future experiments will warrant disregarding these, even if the results they give are not "absolutely correct."

As to the quantity of water actually measured: Plate LXXXI gives the list of the experiments on which Plate LXXX is based. For the results given by Plate LXXX, some of which are presented in Table 69, a high degree of accuracy is claimed. These experimental data cover the limits given both on Plate LXXX and in Table 69. The

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writer is confident that they will give accurate results under conditions "which are met in the operation of a canal system."

Little Additional Cost Brings Better and More Accurate Results.—The small additional cost of "constructing a channel of approach", instead of making "the enlargement of the up-stream portion of the stream bed to a sufficient area" (and Mr. Winsor might well have added to a sufficient depth), will, in the opinion of the writer, be much more than compensated by the additional convenience, accuracy, and satisfaction that will result. Mr. Winsor certainly appreciates the value of maintaining a large velocity of approach in this mountainous country, as does every other engineer who has had experience where streams carry great quantities of silt.

Information Concerning One Particular Weir More Needed Now Than Information Concerning a General Solution.—It is not clear why Mr. King thinks that, in order to be of general application, the solution to the weir problem "must provide for a channel of approach of any cross-section." Of course, conditions will arise which make it desirable to know what quantity of water with a given head is passing over some irregular crest having uncertain contractions. Great care has been taken and much space used to provide, in this paper, a means of measuring the quantity of water flowing over irregular weirs, and the results reached are satisfactory; but, when water is to be measured with a high degree of accuracy, the device to be used must be built for this purpose.

As the suppressed weir can be used under practically all conditions where accurate results are demanded, the writer thinks it better for hydraulicians, who are in a position to do so, to gather more information concerning the discharge over this weir than to attempt to get a "satisfactory general method of solution for all weir problems." Although this general information would be of interest and value, the writer regards the more scientific data as having a greater value.

The Greater the Velocity of Approach the Better.—As water rarely if ever flows over a weir without some velocity of approach, Mr. King's suggestion to secure data for this condition does not appeal to the writer as of great importance. In fact, as the greater the velocity of approach the more ideal is the weir for use where silt is carried in the water, would it not be better to gather information concerning weirs with smaller heights and therefore with still greater velocities of approach?

The Best Standard Weir for All Cases.—Taking into account the quantity and reliability of the data available, the range of heights covered by the series of weir experiments used herein (see list on Plate LXXXI), and the comparatively wide range of heads covered by these experiments, the writer must differ with Mr. Horton, and say that there is a weir "which is the best standard for all cases", and that it

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should be so considered by all engineers until data are furnished that will show some other weir at least equal, in its general application, to that without end contractions.

Simplicity of Method.—Mr. Horton's discussion argues in favor of making "the computation as simple as possible by using the $\frac{3}{2}$ power of the head, instead of causing the head to vary according to some other fractional or decimal power, which would require the use of logarithms, logarithmic paper, or a logarithmic slide-rule for practical computations," and he closes remarking "regretfully, as regards hydraulic science, 'Of the making of many formulas there is no end' ".

Although this paper presents a great array of formulas, these are used, and are intended only to be used, for the purpose of preparing diagrams similar to those shown on Plate LXXX and in Table 69, from which, without calculation, discharges are obtained.

Mr. Smith states that he cannot agree that the diagrams herein presented are "in the interests of simplicity." Then he adds "errors in taking results from the diagrams can easily exceed the discrepancy between the author's and the Francis formulas." The writer believes that Mr. Smith will change his view after examining Table 72. If he takes the results obtained from Plate LXXX and checks them by Table 69, he will probably be surprised at the smallness of the "discrepancy."

Discharge Proportional to Length of Crest.—Mr. Smith says, again, "one assumption made at the outset of the paper is perhaps open to question. It is that the discharge is proportional to the length of crest."

In reply, attention is drawn to the fact that the famous hydraulician, Bazin, made very careful experiments on his two standard weirs, one having a length of 1 m., the other having a length of 2 m. Concerning the results obtained, he writes:

"All needful precautions were taken to insure a flow under exactly the same conditions as over the 2-m. weir. The height of the weir remained the same. It might, however, be asked whether, in view of a somewhat different distribution of the velocities, or of an increase in the effect of friction against the side-walls, the results would be strictly comparable. We have failed to observe any appreciable difference between the two series."

In the new canals of the Hydraulic Laboratory of the University of Utah, the writer hopes to be able to determine whether or not the discharge is proportional to the length of crest, and also if the friction on the side-walls, where the velocity is comparatively small, is a factor that must be taken into account when a high degree of accuracy is desired.

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Mr. Smith asserts, with others, that "the Cippoletti weir discharge is proportional to the length." If this were true in the sense that the same quantity of water passes over each and every unit of length of crest of this weir, it could be used for a dividing device, as can the weir without end contractions. A moment's reflection shows that water flows through the triangular areas outside of the ends of the crest proper; it follows, therefore, that the discharge is not proportional to the length. For this reason the Cippoletti weir cannot be used as an accurate dividing device.

Simplicity of Table 69.—The table of discharges (Table 69) is believed to be quite as simple as the table referred to by Mr. Smith, which he says "is so simple that an engineer is not required to interpret it."

Height of Crest of Cippoletti Weir Omitted.—If the "height" of the crest of the Cippoletti weir above the bottom of the channel of approach is not specified, then the statement made by Mr. Smith that the error, under the three conditions he names, "will be less than 1%" is far from correct.

One Type of Weir Should Be Established by Legislation.—"Exception," Mr. Smith says, "should be taken" to the "proposition to establish one type of weir for universal use by legislation." Every engineer who has been connected with power-plant work, involving, as it does and must, the efficiency of hydraulic motors, has learned that, in order to determine in a satisfactory way the quantity of water used, it is wise, if not positively necessary, to specify how the water is to be measured. When, as is the case these days, "bonuses and forfeits" involving thousands of dollars are based on variations of discharge as small as one-tenth of 1%, as stated by Mr. Williams, it is certainly advisable, by legislation or otherwise, to fix some one standard form of weir measurement. In such a case it is as important, perhaps, to get uniform results as to get results which are accurate.

Mr. Smith says, "The best method for a particular case depends on the surrounding conditions." The writer holds that the best method for all cases is that by which the same quantity of flowing water can be measured again and again by different trained observers, each measurement giving the same result. This is especially true if the result is an accurate one.

That the Utah law establishing uniform cross-sections for roads is an unwise one is a fact that has no bearing on this discussion.

Results Inaccurate When not Based on Experiment.—Can Mr. Smith know, or can he or any one else find out, at any reasonable cost, what degree of accuracy he obtained when measuring water with the special weir described in his discussion? The channel of approach, he says, was "scoured out to a depth of from 1 to 2 ft." Thus great additional uncertainty was introduced into the results.

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Modified Venturi Meter.—The writer agrees with Mr. Smith that "in the western part of the United States it is likely that a modification of the Venturi water meter will come into use to a considerable extent." Experimental data on a device of this sort will be welcomed.

Undershot Weir.—Another device, which may be used with great satisfaction in the near future, is the "inverted weir" without end contractions, that is, a notch in the bottom of a diaphragm across the channel, with contraction at the top only. Through such a notch or under such a "weir", sediment would pass readily. In order to make this structure a very valuable measuring device, it only remains for some one to determine the relation between the quantity of water flowing through the notch and the difference in elevation between the surface of the water above and the surface of the water below the diaphragm.

The Cippoletti or the Suppressed Weir?—Mr. Jarvis writes:

"If a single type of weir were adopted, either the Cippoletti or the sharp-crested rectangular weir without end contractions would doubtless serve best in irrigation practice."

For accurate work, where measurements are to be taken over comparatively long periods and under varying conditions, practically every good reason is in favor of the suppressed and against the Cippoletti weir.

Weir for Quick Rough Work.—A case cited by Mr. Jarvis is a good one in favor of the Cippoletti weir; that is, where a portable device of this sort is used by a ditch rider. When such rough and hurried measurements give the accuracy desired, it may be wise to use the Cippoletti weir.

Measuring Flume and Current Meter.—Mr. Jarvis draws attention to the use of the measuring flume and the current meter. In canals where the slope is slight, these can be wisely used.

Maximum Discharge, Minimum Obstruction.—The writer is grateful to Mr. Jarvis for the clear statement that "the rectangular weir without end contractions provides the maximum width of crest, and the minimum obstruction to the flow in the canal."

Difficulties Encountered in Accurate Measurement of Heads.—The value of this paper is greatly increased by the keen comments of Professor Williams. He points out errors which are introduced into weir work through inaccuracies in measuring heads. He also presents much original matter relating to differences which occur when the same head is measured on the same weir at the same time by different persons and in different ways. He points out specifically the difficulties encountered when an attempt is made to measure heads carefully.

Conclusion.—In conclusion the writer could hardly do better, perhaps, than refer to the six reasons given near the beginning of this

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paper setting forth the advantages in practice of using weirs without end contractions. In the language of Mr. Jarvis, these weirs "provide the maximum width of crest and the minimum obstruction to the flow in the canal." This is a vital matter where streams carry great quantities of silt.

New practical features added since the paper was written are the use of flush-gates in the bulkhead which carries the permanently fixed crest (Figs. 50 and 51); suggestive definite dimensions for weirs to be used for measuring certain quantities of water, and the quantities of concrete required to build the structures recommended (Table 71); and a table giving discharges over all sharp-crested weirs without end contractions within the limits of experimental data (Table 69).

The writer is more strongly convinced than ever that when irrigation engineers and practical hydraulicians have given careful consideration to the matters contained herein they will agree with him that, for general application, where accurate practical information is wanted, and where any weir can be used for measuring water, that without end contractions will answer the purpose best.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.

Discussion.*

By W. W. Follett, M. Am. Soc. C. E.†

W. W. Follett, M. Am. Soc. C. E. (by letter).—Owing to illness and absence from his office, where all his records are kept, the writer can only review the discussion of this paper briefly. The interest shown by the members in the subject has been gratifying. Mr. Follett.

Mr. Landreth brings out quite plainly the relation between the number of shots per square mile and the cost. His figures, however, as would be expected, show that the cost does not increase in direct ratio to the number of shots. With 15 times as many shots as on the work described by the writer, his cost was only 4½ times as much per square mile. Of course, conditions as to brush, etc., may have affected the relative cost of the two pieces of work.

After further study, the writer is inclined to agree with Mr. Brown that he set the maximum errors too high, and will modify his statement on that subject to read:

"In any work where a variation of ½ m. to 1 m. in the relative location of points near together, or 3 or 4 m. in that of those which are material distances apart, can be tolerated, the stadia offers a most rapid and handy method of work."

Beyond this, he cannot go. The Cincinnati surveys must have been most thoroughly controlled by triangulation. Even then, the writer does not see how a map could be made, which would scale so closely

* Continued from February, 1914, *Proceedings*.
† Author's closure.

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under all conditions of weather. He has found that map paper swells when the air is moist and that the swelling crosswise of the sheet may be twice that lengthwise, so that the map is distorted and measurements of long distances cannot be checked to the limit mentioned of "$\frac{1}{80}$ of an inch".

The rod used permitted of accurate readings, under good atmospheric conditions. The work was not done in a slipshod manner, as might be inferred from Mr. Brown's remarks. Certain standards, which were considered proper for obtaining data for a map of 1 in 5 000, were adopted and enforced.

The writer regrets that he did not note the table on page 235 of General Barlow's report, and reproduced by Mr. Smith, as Table 25. He took the distances between monuments from the official report, signed by all six commissioners—three on each side—supposing them to be correct. It is gratifying to know that his chaining was so good.

Two of the Brandis transits used on this Barlow work fell to the writer, and he used them on an extensive survey of the lower Rio Grande in 1897-98. They were a source of constant worry, as the stadia wires would vary slightly in their intercept and in neither was the interval 1 to 100, nor was the middle wire half way between the other two. Frequent interval determinations were necessary.

The use of the needle, as suggested by Mr. Smith, does not appeal to the writer, except for very rough preliminary surveys. He does not believe that any man can read a needle closer than to 5′ of arc, and his error may be 10′, and an error will be introduced which is many times that which arises from the neglect of $f + c$. The diurnal variations will also affect the needle readings. This may do for very rough work, but not where a fair degree of accuracy is desired.

It was not the writer's intention to intimate that he sent instruments to the makers for adjustments other than that of the stadia wires. In fact, he has never sent an instrument to the maker for any adjustment. If the stadia wires did not subtend 1 in 100, he used a reduction table. He merely suggested the expediency of having wires reset.

The point in the paper most criticized was the ignoring of $f + c$ and the absence of interval determinations. The writer should have stated that the intervals of the transits used on the stadia work were tested at the beginning and at the close of the survey, and that one, with which the instrumentman did not get as good closures as the other men, was tested two or three times during the progress of the work.

Assuming that $f + c$ is to be ignored in the field work, the writer would enquire how a man is to proceed to make a reduction table when, the hubs having been accurately set by steel tape 50 m. apart and the transit set up $f + c$ (which was, for the transits used, if the writer's

memory serves him, about 0.3 m.) back of the first hub, the instrument-man states that he reads exactly 50 cm. on the 50-m. hub, exactly 1.00 m. on the 100-m. hub, etc., and when the other men present, including the writer, check these readings? Three of the five transits used on the survey gave this result. It was known that the intervals of the other two were not 1 in 100 but, as they were not to be used on stadia work, their intervals were not determined. Mr. Follett.

As for the ignoring of $f + c$, the results given in the paper, in the writer's opinion, are sufficient to show its expediency.

The writer agrees with Mr. Brown that the quoting of cost data on any work, stadia or other, is a dangerous proceeding unless all the conditions are fully stated, and that even then it may be misleading. It was the intention to compare the cost of this with similar work. He cannot, however, imagine any case where he would feel justified in spending from $1 000 to $2 000 per sq. mile, or even the first amount, on a stadia survey. If property is so valuable that such an expense in making a survey of it is justified, then some method more accurate than stadia should be used.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

BY MESSRS. FREDERIC V. ABBOT, J. F. RAMSBOTHAM, AND HARRISON SOUDER.

Mr. Abbot.

FREDERIC V. ABBOT, M. AM. SOC. C. E. (by letter).—Mr. Rippey's discussion† is exceedingly interesting, because it carries to much further length a method adopted by the writer in 1899, in constructing a cut-off wall of moderate depth at Lock and Dam No. 2, Mississippi River, between St. Paul and Minneapolis. At this locality the material forming the bed of the river was a very hard combination of clay, sand, and limestone fragments, mixed with a few granite boulders, overlying the well-known, friable sand rock of this part of the Mississippi Valley.

Fig. 19, shows the nature of this material, which underlaid the lock floor and the lock walls. A description of the difficulties encountered in handling this material, which is quickly eroded by rapidly flowing water, and of the methods adopted to overcome them, is given in the following extract quoted from the writer's Annual Report to the Chief of Engineers, U. S. Army, dated July 17th, 1900:

"The seams were all originally water-bearing, and only became dry after long pumping had drained out all the water tributary to them. At a lower level these seams run out into the bed of the river, and in this way passages for water are opened between the river bed and the interior of the cofferdam, many feet below the original bottom of the river at the point where the cofferdam stands. It had been realized from the start that such seams were to be encountered, and to avoid opening up too great a flow at any one time a novel form of cut-off wall was adopted after prolonged study. A slot in the sand rock is made, 2 inches wide and 12 feet long, parallel to the river wall and lying between two longitudinal foundation timbers, by jetting

* Continued from April, 1914, *Proceedings*.
† *Proceedings*, Am. Soc. C. E., March, 1914, p. 779.

Mr. Abbot.

a series of intersecting vertical holes 10 feet deep and breaking off any projecting edges between the holes. Most of the eroded material is brought up out of the slot by the jet, and the rest is easily pumped out with the ordinary Edson diaphragm pump and two men. In the usual case this leaves a perfectly clean slot with a considerable flow of water pouring out of it, the water coming from the underground seams which connect with the river. Into this slot a diaphragm made of tongued and grooved three-quarter-inch boards, in two layers breaking joints, is lowered by hand. Three vertical 1½-inch iron pipes, extending down into the slot only a few inches, are inserted, one at each end and one at the middle of the slotted section. A steam pump connected to one of these pipes is now started just fast enough to take care of the flow from the slot without letting it run over, or draining it below the level of the surface more than a few inches, and concrete is rammed into the space above the slot and between the foundation timbers above referred to, which have been firmly drift-bolted down to the sand rock before the slot was cut. As soon as the concrete has set—that is, in about twenty-four hours—the pump is stopped, and water at once flows out of all three pipes. Additional lengths of pipe are screwed on the three till the flow is stopped—that is, till the tops of the pipes are at a higher level than the head due to the pressure in the seams. Thick grout is now poured into smaller pipes passing through the first pipes and reaching to the bottom of the slot. As the latter fills with grout the water is forced back into the seams. The small pipes choke after the grout rises a couple of feet above their lower ends, and they are then raised till they again flow freely. When the slot is full they are removed and the larger pipes are kept filled to the top with the grout as long as any more can be poured in. After about twenty-four hours all the grout has hardened, the pipes are removed, cleaned out, and used over again in another section of cut-off. (Photographs Nos. 7, 8, and 9* show an experimental section of such a cut-off wall which was put in at a high level in the lock pit and then carefully excavated.) The perfect grout filling of the most minute

Note on Fig. 19.—The extreme friability of the material is shown. The method of supporting the timbers on this unreliable material is shown at the top of the picture. The timber is wedged up on wooden blocking so as to leave a space of about 2 in. between it and the surface of the sand rock. The anchor-bolts are then driven and the wedges removed, leaving the timber supported on the anchor-bolts. The surface of the sand rock is then thoroughly washed with water from a hose, all loose matter is removed, and the space between the fresh sand rock surface and the timber is filled with good concrete rammed in place with heavy sledges and steel followers.

Note on Fig. 20.—The two large timbers between which the concrete is seen represent foundation sills. They are fastened to the sand rock every 4 ft. by the 1¼-in. drift-bolts, of which one is seen extending down into the wooden pin, a part of which has been removed. These bolts extend 3½ ft. into the wooden pins, and require a force of more than 9 tons to withdraw them. The wooden pin to the left is shown encased with grout, as it was when the sand rock was carefully removed from around it. The narrow white column about half way between the two pins is the end of the cut-off wall as it appeared when the first 2 ft. of the sand rock was removed. The white marks, looking like splashes of whitewash, are really thin filaments of grout which had penetrated the seams in the sand rock, showing that the cut-off wall is not only a thin impervious vertical curtain in the sand rock, but that the latter itself is solidified for at least 2 ft. on each side. The board in a horizontal position under the concrete is used to prevent the fresh concrete from falling down in the slot alongside of the board curtain, and thus preventing the grout from having a thorough subsequent circulation.

* Reproduced herein as Figs. 21, 20, and 22, respectively.

FIG. 19.—VERTICAL FACE OF SAND ROCK ON THE LOWER SIDE OF THE EXCAVATION FOR THE UPPER LOCK GATE AND CULVERTS.

FIG. 20.—EXPERIMENTAL SECTION OF CUT-OFF WALL, PARTLY EXCAVATED, TO DETERMINE WHETHER THE GROUT FILLED ALL CREVICES, ETC.

cracks in the sand rock is surprising. An absolutely tight cut-off wall from the bank above the lock to the outer toe of the river wall, and under this toe for its full length and back to the bank below the lock, extending to a depth of 23 feet below dead low water in the river, or deeper in that part under the river wall, is thus secured. Mr. Abbot.

"In some cases no subterranean seams are encountered, and then the wooden diaphragm is inserted and the grout at once poured in around it through pipes leading to the bottom. Experience has shown that in such cases time is a vital element. A slot which does not develop the least flow at first, has in several cases developed very great and troublesome flow in twenty-four hours if not grouted. Most trouble was encountered at the upper end where a very large seam directly connecting with the river had to be cut off. This seam extended nearly horizontally under a space fully 30 feet wide by 80 feet long, and every section of cut-off wall intersected it. Grout poured into one section would pour out of another 30 feet distant. As these underground connections were not suspected, the quantity of water which poured out of three openings into this seam soon exceeded the capacity of the pumps. All arrangements for grouting two of the three sections were completed except the stopping of the flow from the pipes, which in this case were 6-inch in diameter and were running full. It is believed to be unsafe to try to stop suddenly such a flow for fear of blowing up the lock floor. The cofferdam was allowed to fill, thus gradually stopping the flow from the seam, 6-inch risers were screwed into the now submerged overflow pipes and both sections were grouted at the same time from staging erected above the water level. Twenty-one barrels of dry sand cement were required for the grout to fill the slots and seam. In three days the cofferdam was again pumped out and the two springs were found stopped entirely. The third was really outside of the lock walls, being in a section of slot that had been put in to make connection with the cut-off wall under the dam. It was successfully separated from the cut-off slot under the lock wall, which has since been easily grouted."

In the preceding paragraph mention is made of sand-cement. In view of the high freight rates existing at that time between the American Portland cement factories and St. Paul, sand-cement was used in the construction of the land wall of this lock. To insure getting exactly the proportions of sand in the sand-cement, a tube mill was purchased and the cement (Saylor's brand) was ground at the lock site with a very excellent silicious sand found in the vicinity. The concrete made with this sand-cement has stood the extreme temperature conditions at St. Paul, 105° in summer to 40° below zero, Fahr., in winter, for the past 14 years in a surprisingly satisfactory manner. The river wall was built of Saylor's cement, used in the same proportions as the sand-cement in the land wall. A very sharp comparison of the two kinds of concrete has thus been secured. Although the writer has not been able to inspect the work, he has been told by those who have, that the concrete in the land wall is, if anything, superior at the present time to that in the river wall. For

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the land wall, the cement was ground with an equal proportion of sand, by volume, and the sand-cement thus produced was used in the same ratio to unground sand and broken stone as if it had been regular Saylor cement.

In connection with the construction of this lock, a great deal of grouting of seams similar to that described was necessary, and in this work great economy resulted from the use of sand-cement in place of undiluted Saylor cement. Experiments were made in order to determine whether sand-cement applied in the shape of grout would harden. A vertical glass tube, 4 or 5 ft. long and about 1 in. in diameter, was plugged at the bottom and filled with a 1 to 1 sand-cement grout and allowed to harden. The tube was broken in 3 days, and it was found that the lower part of the grouting in this pipe was as hard and sound as that at the top. When ordinary sand was mixed in the same proportion with undiluted Saylor's cement, kept thoroughly stirred up, and poured into a similar tube, the sand settled to the bottom very rapidly, leaving a creamy grout with hardly any sand grains in the upper part of the tube. Under these conditions the bottom 2 ft. never set, though the upper 2 or 3 ft. set about as hard as the sand-cement grout did in the other tube. The reason is plain. The specific gravity of cement clinker is not widely different from that of silica. When the silica is in large grains, and the cement clinker is ground fine to make a Portland cement, the two materials separate in water, the larger particles settle quickly and reach the bottom first, and the finer particles, which settle less rapidly, remain near the top of the column. Since making this experiment the writer has always had considerable doubt as to results of grouting with Portland cement stirred up with ordinary sand, but he has no doubt of the effectiveness of grouting with sand-cement ground 1 part of cement to 1 part of silicious sand, provided the grinding is carried far enough to make the sand as fine as ordinary Portland cement. From Mr. Rippey's description, the writer supposes that no sand was incorporated in his grout. If sand-cement had been used, it would seem, therefore, that the cost of grouting might have been somewhat reduced provided the freight rates to the Estacada Dam were high enough to make it pay to install a cement-grinding tube mill.

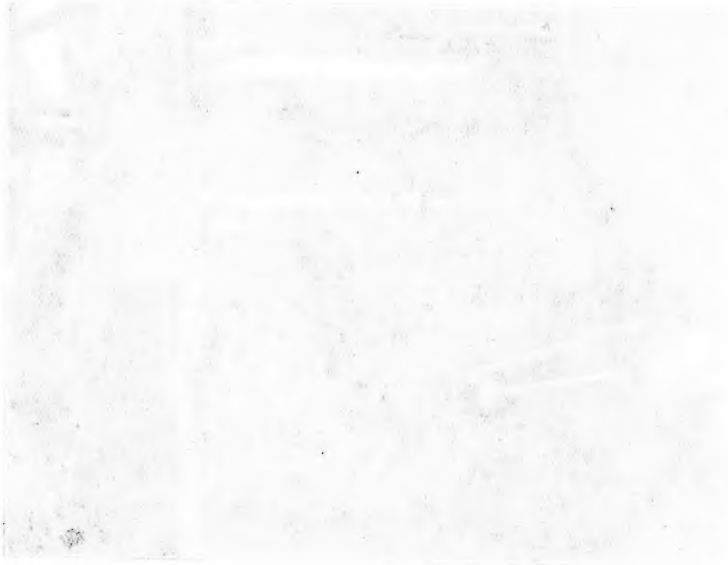
Note on Fig. 21.—The complete covering of the end of the wooden curtain by the grout is well shown, as well as the way in which the latter filled every irregularity in the end of the original slot. The grout was about 1 week old, and was at the time firm and hard. It was composed of sand-cement ground in the proportion of 1 sand to 1 Saylor's Portland cement. The extreme wetness of the sand rock is well seen in the pond of water at the foot of the section. Though there was no regular flow, the water was oozing from all the pores of the face exposed by the excavation.

Note on Fig. 22.—At the top, the grout covering the end of the diaphragm has been removed to show the boards, and, in the middle of the plane parallel to the diaphragm, the grout has been scraped off to show the flat side of the boards. The rest of this plane is left as it stood when excavated, showing that the grout absolutely holds the sand rock to the boards. The irregular projections at some points in this plane show where there was originally a seam in the sand rock, which has been filled by the grout.

EXPERIMENTAL SECTIONS OF CUT-OFF WALL.

FIG. 21.—WHEN EXCAVATION HAD PROGRESSED TO A DEPTH OF 6 FEET.

FIG. 22.—A SECTION PARALLEL TO THE DIAPHRAGM OF BOARDS, AND A FEW INCHES, ONLY, FROM IT.



J. F. RAMSBOTHAM, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with great interest and is of the opinion that it will be of value to the entire Profession. He would like to draw some conclusions based on his experience gained on the Fremantle Graving Dock.* Mr. Ramsbotham.

In the first instance he differs with the author in regard to the method of starting with a low pressure of 25 lb. and ending with a pressure of 250 lb. per sq. in. Surely the pressure should have been higher in the initial stages, for, with the depth of hole in use, there was no fear of doing damage, and, at the same time, a better and more pronounced penetration would have been assured.

A pressure of 250 lb. per sq. in., or 16 tons per sq. ft., is certainly very high, and the quality of the rock must have been exceedingly tough to have withstood it. The danger of using such a pressure is considerable, but risk of damage would be lessened by having other holes drilled in the vicinity of the grouting, such holes acting as a safety valve.

The author's summary of the advantages reaped is somewhat disappointing.

From the writer's experience, cement grouting can be relied on to form a cut-off from percolation water, and this experience was gained by witnessing daily a material decrease in pumping.

The holes should be staggered, the front row being 6 ft. from center to center and the back row 3 ft. back from them. The writer considers it preferable to drill the front row first and then check and test the results by the back row, but it is quite impossible to lay down any hard and fast rule. Each case must be judged on its merits.

The whole question of efficiency is problematical, and for that reason, before adopting cement grouting and giving a positive opinion, an actual inspection of the site is desirable.

As regards fortifying bad ground (such as a gravel bed) to withstand a heavy statical load, the writer has no hestitation in saying that by cement grouting this can be done economically and safely, there is no excavation to make and replace with concrete, and no pumping.

HARRISON SOUDER, M. AM. SOC. C. E. (by letter).—It is a fact that very little has been published in regard to the treatment of foundations by the use of cement grout. The interest manifested in the Estacada work causes the writer to regret deeply that his experiences in this line some 13 years ago, were not given to the Profession at the time. However, the identical method of securing a tight cut-off wall—by injecting a cement grout under pressure through drill holes, as described by Mr. Rands—was used by the writer in the construction of the Hinckston Run Dam at Johnstown, Pa., as long ago as 1901. Mr. Souder.

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1942.

Mr. Souder.

The fact that this method of grouting was used in the foundations of this work prior to the Estacada operation is brought out in a brief description of the Hinckston Run Dam published in 1909 by the late James D. Schuyler, M. Am. Soc. C. E.*

At the time this method was adopted at Johnstown, the only information the writer had on the subject was that gained from watching the grouting of the tunnel backing of the deep sewers built in connection with the Pennsylvania Avenue Subway and Tunnel and in closing leaks in the Torresdale Conduits, in Philadelphia; and especially from a report† by F. V. Abbot, M. Am. Soc. C. E., Major (now Col.) Corps of Engineers, U. S. A., Engineer in Charge of Improvements of the Mississippi River at St. Paul and Minneapolis. The report describes an extremely interesting method of forming a tight cut-off wall by using cement grout, exactly after the method suggested by Mr. Rippey on page 779‡ and illustrated by Fig. 16, except that the diaphragm was made of wood and the grout was applied under hydrostatic pressure sufficient to overcome the head of the water flowing through the rock seams.§

As the work at Johnstown was in the seamy shales of the soft-coal formation, a detailed description of the cut-off wall may be of value even at this late day. To add to the clearness, and as a matter of record, a brief description of the dam is given.

The Hinckston Run Dam.

In 1900 the Manufacturers Water Company, after a comprehensive study of the problem of increasing the water supply of Johnstown, Pa., and the rapidly expanding plant of the Cambria Steel Company, determined on the construction of several large reservoirs and pipe lines.

In view of the frightful calamity of 1888, caused by the breaking of the South Fork Dam, there was naturally much opposition to the construction of any new dams above the city. Under the direction of Mr. John Birkinbine, as Consulting Engineer, the first of the proposed reservoirs was located 5 miles from Johnstown, on Hinckston Run, a stream which flows into the Conemaugh a short distance below the famous Stone Railroad Bridge. All the streams in this district are essentially mountain streams. The valleys are narrow and the hillsides steep and rocky, and therefore the run-off is rapid. The average yearly rainfall is 36 in., but it is not well distributed, the major part coming down in the winter and spring; though some extremely heavy and sudden downpours occur in the summer.

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* "Reservoirs for Irrigation, Water Power and Domestic Water Supply," p. 518.

† An abstract of this report was published in *Cement*, January, 1901.

‡ *Proceedings*, Am. Soc. C. E., for March, 1914.

§ The reader is referred to Col. Abbot's discussion, which contains an extract from his report, and several illustrations.

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The original Hinckston Run project called for an earth dam, 60 ft. high, to retain some 400 000 000 gal. of water, with a depth of 45 ft. at the breast.

The intention, as stated, was to build a dam 60 ft. high, with a clay core, but, as an unlimited quantity of cinder from the steel plant was available, it was decided, after the work was started, to use this as backing for the dam, in place of earth, and eventually to fill the whole valley below with this material, thus rendering the structure practically unbreakable. In view of this and the additional expense incurred in making the cut-off tight, the proposed height of the dam was increased to 80 ft., and later to 85 ft., above the original creek level. This gave a total maximum height above the bottom of the core-wall ditch of 112.8 ft., a depth of water at the breast of 73½ ft., and a capacity of 1 100 000 000 gal. The lake thus formed is 1¼ miles long. The water-shed above the dam is 10.75 sq. miles.

With the consequent great increase in pressure, extra care was needed, and every precaution was taken to make the structure water-tight.

The site selected for the dam required a rather long breast, but gave an excellent opportunity to construct a spillway with natural rock bottom, and was close to several large deposits of excellent clay lying within the reservoir site.

Work on this dam was started in the fall of 1900. The writer took charge as Resident Engineer and Superintendent of Construction early in May, 1901. At that time the dam site had been cleared of timber and partly stripped, and a small section of core-wall ditch had been excavated to a depth of 12 ft.

The first work done was to strip the surface of the dam site of all soil and vegetable matter. This was generally 3 ft. deep, but over a large area, near the creek channel, it was necessary to remove material to a depth of 8 or 9 ft. owing to numerous beds of muck, old logs, etc. All this material was deposited in the rear of the embankment proper. The remainder of the reservoir area, some 105 acres, was cleared of all timber and brush, but the surface soil was not removed, as the water to be stored was intended for mill purposes only.

Work on the dam proper was suspended on rainy days and during the winter.

The cross-section of the dam as built is shown by Fig. 23. The lower inner slope is 1 on 2¼, with 4 ft. of puddle and 24 in. of cinder rip-rap. The slope above the berm is 1 on 1¾ with puddle lining diminishing to 2 ft. thick at the top. The facing is hand-laid stone paving. The puddle wall is 16 ft. thick at the top of the concrete core-wall, and diminishes to 4 ft. at the top of the dam.

Early in July, 1901, when the core-wall ditch had been carried down to hard rock at what was thought to be the proper depth, some test

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holes were bored through the bottom to determine the character of the rock below. This disclosed a layer of hard sandstone a few feet down, with considerable water flowing below and above it. It was decided to deepen the ditch considerably, in order to get below any rock strata that might come to the surface within the flooded area, and to substitute a concrete core-wall for the clay one originally proposed.

An air compressor plant was installed. This was a 14 by 18-in. Ingersoll-Sergeant machine capable of driving two rock drills and six pneumatic rammers. These rammers were used in tamping concrete and also in puddling clay in such places as could not be covered by a 10-ton steam roller which was also supplied at this time in place of the 3-ton horse roller in use.

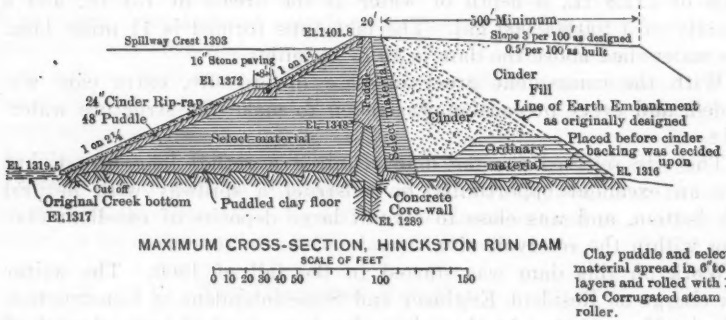
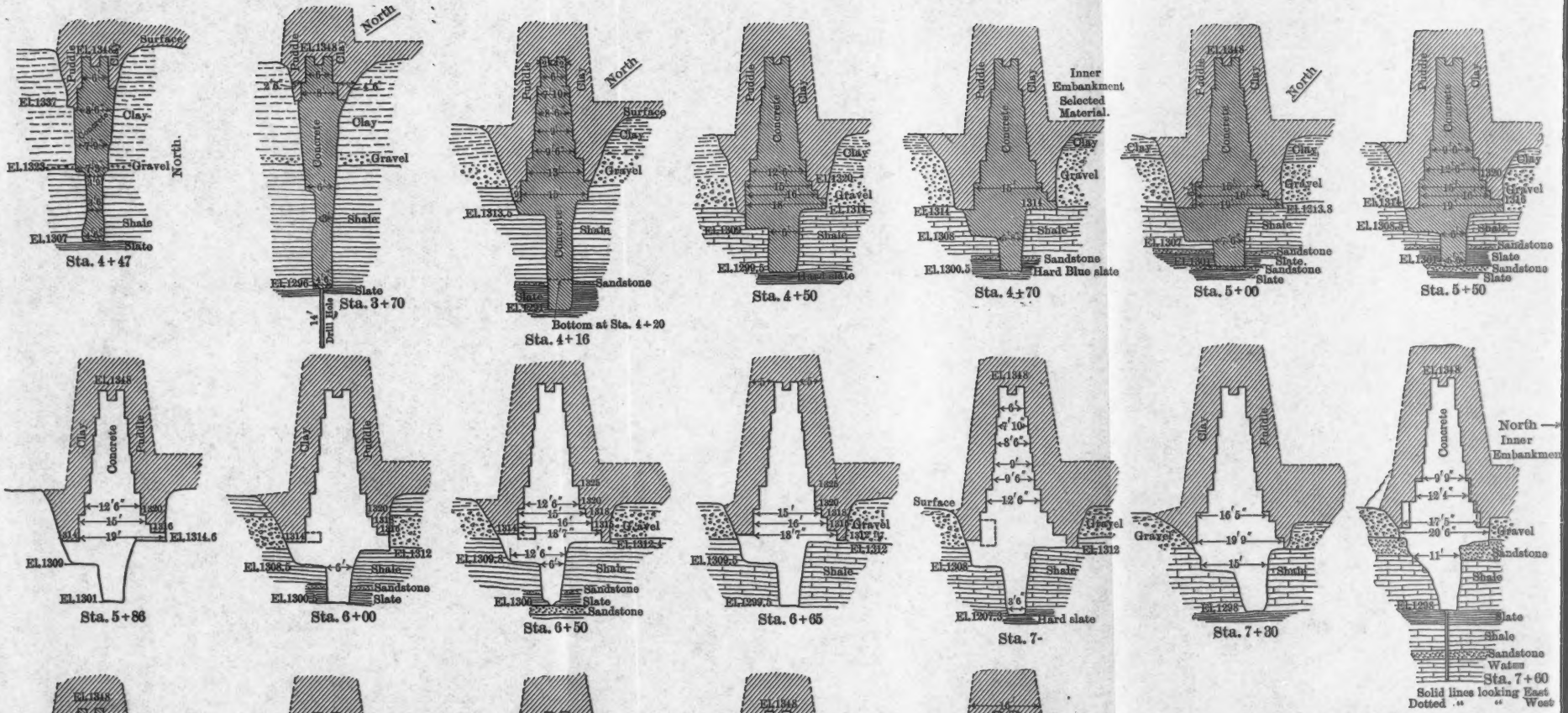


FIG. 23.

The finished ditch averaged in depth 25 ft. below the grubbed surface in the valley, but reached 50 ft. at the ends. The shale was excavated with picks, but the harder rock was loosened with light charges of dynamite, care being taken not to shatter the foundation or open up the seams. Plate XII shows typical sections through the core-wall and ditch, the variable character of the strata cut through being indicated. The advisability of cutting off the underflow to as great a depth as possible was realized, and it was determined to remove the shale down to the sandstone and try to cut off the flow below by forcing in cement grout under air pressure.

Drill holes, 2 in. in diameter and from 10 to 16 ft. deep, were drilled through the rock, averaging about one hole per linear foot across the valley. Iron pipes, 2 in. in diameter, 18 in. long, and threaded on one end, were cemented into these holes. Portland cement grout was poured into them and then air at a pressure of from 30 to 60 lb. was applied.

The first holes were approximately 6 ft. apart. They were drilled generally 10 ft. below bed-rock. The first hole drilled was marked No. 1 W., the second No. 2 W., 5 ft. distant, the third No. 4 W., 17 ft. distant.



Note- Drill holes 2" diam. in bottom of ditch; 10 to 16 ft. deep between Stations 3+60 and 8+00; and 6 to 10 ft. deep between Stations 8 and 8+70. Cement grout forced into these holes under 60 lb. air pressure.

CROSS-SECTIONS

CORE-WALL AND DITCH

HINCKSTON RUN DAM.

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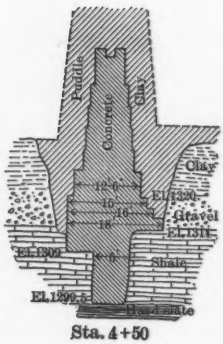
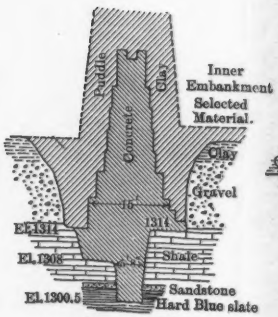
Bottom at Sta. 4+20
4+16



Sta. 4+50



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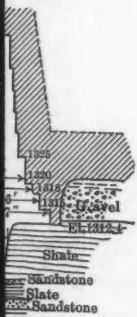

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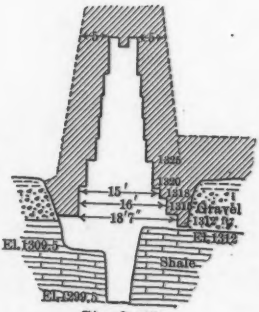

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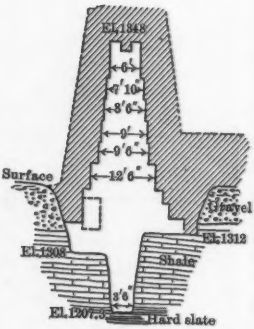

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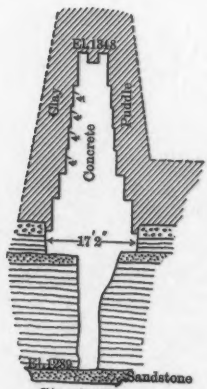

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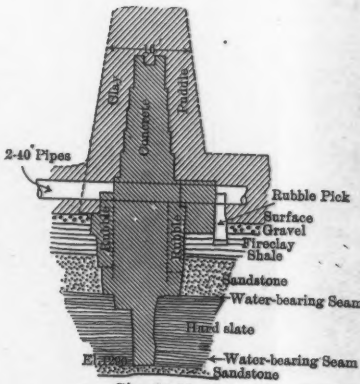

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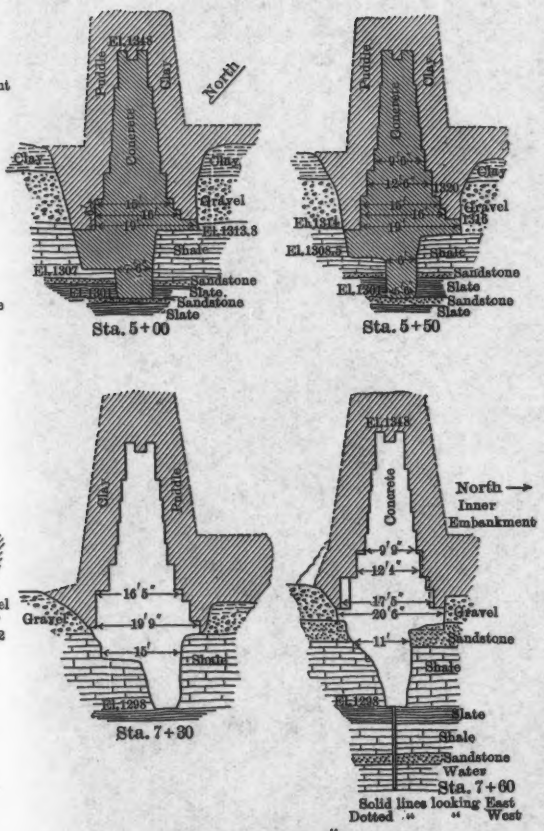

Note- Drill holes 2" diam. in bottom of ditch; 10 to 16 ft. deep between Stations 3+60 and 8+00; and 6 to 10 ft. deep between Stations 8 and 8+70. Cement grout forced into these holes under 60 lb. air pressure.

CROSS-SECTIONS
CORE-WALL AND DITCH
HINCKSTON RUN DAM.

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After No. 1 was grouted, a small pit, or sump, was excavated through bed-rock and into the shale below. Signs of grouting were visible 2 ft. below the sandstone, which was encouraging. After some twelve holes had been drilled within a distance of 66 ft., an effort was made to determine the location and extent of the water seams below bed-rock by using a solution of permanganate of potash. In a letter to Mr. Birkinbine, dated July 22d, 1901, the writer described this test as follows:

"In the main core-wall ditch I made some tests to determine the location and length of seams under bed-rock. Mr. Hyde, the Cambria Chemist, brought some potassium permanganate out. We put several buckets full of the solution into the receiver and connected it with hole No. 3 West and then turned on the air. The color showed in the sump and in holes Nos. 4 and 5 and slightly in No. 6, entering the last hole at least 2 in. from the surface. This showed that 2 in. and 3 in. of shale had not been stripped from bed-rock, also that the seam extended from No. 3 hole beneath bed-rock to the sump. This test was not altogether satisfactory, so we put a pipe in No. 6 W. hole, from which a strong clear stream of water flows. Put some permanganate in the tube and connected the air hose directly to the tube. The color showed in Nos. 4 and 5, in fact, as far as the sump. About this time the pump valve got out of order and the water rose in the trench, covering most of the holes. Then I put on full air pressure, and after a time it began to appear all along the trench bottom from the sump to hole No. 12. Up to No. 10 it was very distinct, at Nos. 11 and 12 the effect was not quite positive. Keeping the pressure on, I had the sump drained, and found the air blowing in at the bottom and on the far side and lower side.

"Evidently there are seams underlying the length of bed-rock. The small *y*'s on the following diagram [Fig. 24] indicate where the air blew out."

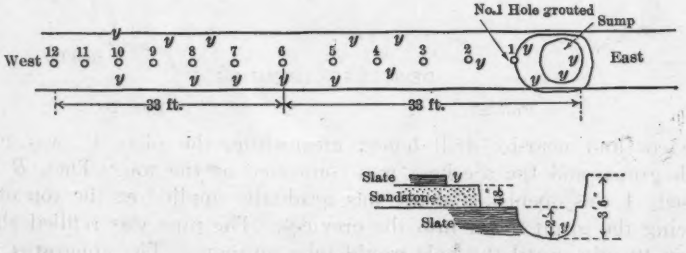

FIG. 24.

Fig. 25 is a sketch of the first contrivance or receiver devised for applying the grout. It was a cylinder, 8 in. in diameter and 6 ft. long. A screw flange was provided at top and bottom, and a steel head-plate was bolted to each end, with rubber gasket packing. The top bolt holes were open to allow quick removal of the lid. A 2-in. pipe with plug cocks was provided at the top and bottom. With a short hose, the cylinder was coupled to the pipes in the holes. The

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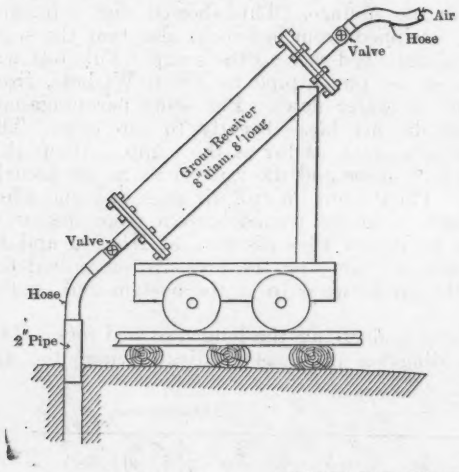
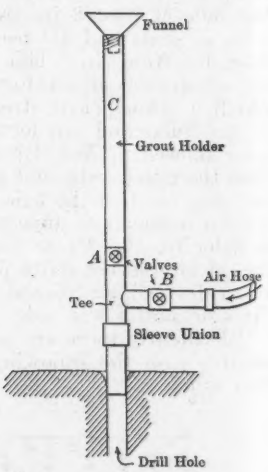
cylinder was filled with grout; the valve was opened; the grout ran into the drill holes, and air pressure was then applied at the top. The contrivance was mounted on a truck running on a track in the bottom of the ditch. After trial it proved to be too slow and cumbersome, and another method was devised and operated satisfactorily. Fig. 26 is a sketch of this final arrangement, and Fig. 27 shows it in operation.

The method of grouting was as follows:

A 1-in. pipe, long enough to reach to the bottom of the hole, was inserted and air was applied to blow out water and dirt. Then a tee and the pipe, *C*, were attached to the tube in the drill hole with a sleeve union, as shown. The cock, *A*, was closed, the cock, *B*, was opened, and air was applied, thus forcing the water out of the hole and into the





DEVICES FOR GROUTING

FIG. 25. FIG. 26.

crevices and near-by drill holes; meanwhile, the pipe, *C*, was filled with grout, and the air hose was connected at the top. Then *B* was closed, *A* was opened, and air was gradually applied at the top of *C*, forcing the grout down into the crevices. The pipe was refilled about every 10 min. until the hole would take no more. The apparatus was then removed and a cap was screwed on the tube in the grouted hole. After a given length of ditch was grouted in this way, and sufficient time had elapsed to allow the grout to set, test holes were bored within the grouted area and the process was continued until there was no indication of water flow below the bottom. The greater part of the bottom was grouted successfully in this way, but, as explained later, the grouting scheme was abandoned where the core-wall ditch entered the side-hills.

FIG. 27.—METHOD OF GROUTING UNDERLYING SEAMS IN CORE-WALL DITCH. GROUTED HOLE WITH CAP, IN FOREGROUND. CEMENT PLASTER ON SIDE-WALLS OF DITCH.

Descriptive of this work, the following extracts from reports by the writer, made at the time, may be of interest: Mr. Souder.

EXTRACTS FROM REPORTS ON GROUTING TESTS.

July 12th, 1901.—"Finished drilling second drill hole. Strip rock near first hole. No signs grout at 3 ft. distance."

July 13th, 1901.—"Excavated to hard rock and then drilled holes every 6 ft., 10 ft. deep. (Later, 16 ft. deep.)

"Cut away 18-in. layer of rock to within 2 ft. of first drill hole grouted and discovered evidences of grouting. Two of the new drill holes flowed water, which on test comes mainly from within 3 ft. of surface of rock."

July 17th, 1901.—"Test of setting of grout; after 2 hours the neat cement grout hardly set enough to resist anything like a current of water. Addition of sand helps it. Recommend quicker-setting cement."

August 3d, 1901.—"There is a thin layer of shale and slate on the sandstone at Hole No. 11 E. becoming thicker to the east at Hole No. 20; the sandstone is 6 ft. below the shale. This explains why water is found at two levels. On No. 19 E. we had water at 4 ft. and again at 15 ft. To-day grouted several holes. The first was No. 13 W. From this hole a fairly strong stream was flowing out of tube 12-in. high. I first forced air in the hole without cement. Air bubbled out of Hole No. 11 W. very weakly but quite strongly from Nos. 12 W., 14 W., 15 W., and 19 W. Made a smooth-flowing grout of 2 bags cement, 1 bag very fine screened sand, and 3½ buckets water. After the air had been on 30 min. a strong cement color appeared in Hole No. 15 and weaker color in Hole No. 14. Kept pressure on for 1 hour. The grout appeared rather soft in the hole, and after a time a very small thread of water came through it. Flow of water from No. 12 did not diminish appreciably, but carried slight cement color. Difference in level of grout in cylinder, before and after, 5½ in. Next grouted No. 11 W., forcing in 3¼ in. of grout after ¾ hour pressure, there was no perceptible difference in flow from adjoining holes. There was no water flowing from No. 11 before grouting. Hole No. 8 took 2½ in. of grout after ¾ hour. No water flowing from the hole before grouting."

August 6th, 1901.—"In the core-wall ditch we have now 13 holes grouted. To-morrow I will move the drill back and put in some intermediate holes to test the grouting. I must arrange in meantime to drain the east end of the core-wall ditch in order to uncover the sandstone, which dips rapidly into the hill. The shale, being more than 6 ft. thick, is giving considerable trouble to the drillers, so think it advisable to suspend drilling in the east end of the core-wall ditch until the sandstone is uncovered. The following is a description of grouting of holes mentioned in letter of last evening:

"Hole No. 4 W.: water standing in tube about 1 ft. high. Cylinder with grout placed and air on for 1 hour. Valve on tube closed for ¾ hour longer. Hole contained 2 ft. cement in bottom, with water above. Rammed thick mortar into tube and put on pressure again for ½ hour, 8½ in. of grout went into the hole.

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"No. 2 W.: no water flowing from this hole. Connected air hose directly to the tube, forced the water out of the crevices. Then poured in two buckets of grout (1 cement, 2 sand) after having air pressure for ½ hour, the grout had settled in the hole 2 in.

"No. 12 W.: strong stream water flowing from this hole at beginning. Forced water out, the air coming out at the sump near hole No. 1 W. Put in one bucket sand and cement dry and put on air for 10 min., after which time the hole was empty. Then put in second bucket dry grout and put on air. This stopped air from coming out at the sump. The hole was practically empty. Then put in a bucket (3 gal.) of wet grout and put on air for 15 min., which left the hole plugged. Hole No. 7: no water from this hole at beginning. Pumped water out by hand-pumps. Put in a bucket wet grout, and after pressure on 15 min. the grout had settled 2½ in."

Again, on September 9th, the writer stated:

"The core-wall ditch, where the pipe line crosses, is coming into a very hard and tough stone, massive, with little evidence so far of stratification or cleavage planes, and but little water.

"In the main core-wall ditch appearances are not so pleasing, I have had four test holes driven.

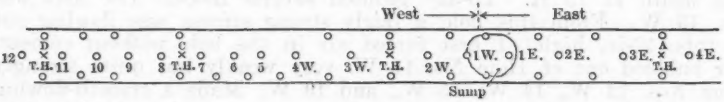


FIG. 28.

They are marked A × T. H. [on Fig. 28]. There was a considerable flow of water from all of them; least from B, and the strongest from C and D. Water was struck in all these holes between 11 and 16 ft. Putting air on at A, it came up at the sump, also through crevices in the bottom of the core-wall ditch near A, also in test holes, B and C. Test hole, D, was not finished at the time. Hole A took 8 buckets of grout; Hole 3 took 3 buckets.

"When air was on C air came out rather strongly at the sump, and at test hole, D, and from bottom of core-wall ditch between holes, T, and 6 W., 7 and 8 W., and 8 and 9 W. After putting in 4 buckets of grout and allowing to stand some time, cement got hard in the valve. Meanwhile we grouted test hole, D, took valve off C and found grout quite wet in C, and began to be forced out by air from D. Put long tube on C and filled with grout and applied air again on C. Then went back to D and fed it more grout, giving it in all 8¼ buckets. Air ceased to bubble up elsewhere.

"It would appear from these tests that we have a fairly tight bottom down to 10 ft. depth, the depth to which the middle holes were drilled. There is, no doubt, however, a considerable flow of water at 10 to 16 ft. depth.

"To expedite the work of drilling in part, and mainly because I think we will get better results in this work, I abandoned the old method of diamond spacing for the bore holes and started this after-

noon to drill the holes right in the middle of the core-wall ditch and spaced but 2 ft. apart. This will give us the same number of holes for a given length as previously, but, being closer, they are more likely to give us a continuous cut-off wall. At the same time, four holes can be drilled from two settings of the bar, as against three settings required for the old system." Mr. Souder.

This is shown by the sketch, Fig. 29.

"First position of bar, *A*, second, *B*. First position of drill on bar, *a*, second, *b*. Simply turn clamp over and reverse the drill in the clamp. I think this will be more satisfactory. Let me have your opinion."

To this Mr. Birkinbine replied:

"I am in receipt of your letter of yesterday, and read with interest your memoranda concerning the test holes. Remembering the fact that the first test holes were only 10 ft. deep, and we got more water in the deeper holes, your results would indicate to me that the section covering the shallow holes had probably been grouted about to their bottom, and that what you are encountering now is a practically continuous stratum below this 10 ft.

"I am inclined to believe that the appearance of air in the floor of the core-wall ditch is attributable mainly to crevices in the upper stratum, where the weight of the material is insufficient to resist the pressure. I think your plan of reversing the bar so as to drill two holes from one setting is a good one, but I am inclined to believe you can afford to separate the holes by more than 2 ft. Of course the holes from one setting will be but 2 ft. apart, but I think there could be an interval of 3 or 4 ft. between one pair, and the next pair. As to drilling these holes in the center of the core-wall ditch I should look for better results if, whenever a longitudinal crevice is evident, you drill close to this, *i. e.*, I would put the holes either to one side or the other of the core-wall ditch, according to the location of these fractures."

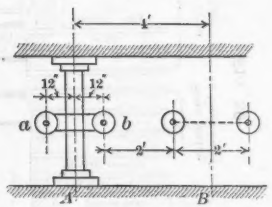


FIG. 29.

Neat cement grout was used after this, the sand being omitted. The proportion was 1 bbl. of cement to 75 gal. of grout.

At the east end three large vertical open crevices were uncovered. (Fig. 30.) The flow from these was so considerable that it was necessary to put in a force pump with a 4-in. suction and a 3-in. discharge, in order to keep the ditch clear. As the grouting method, after continued trials, did not give satisfaction at this end, it was decided to take up the bottom until the principal water strata were reached. This was done, and concrete was placed in the ditch, all the walls being plastered first with a rich cement mortar worked in

Mr. Souder.

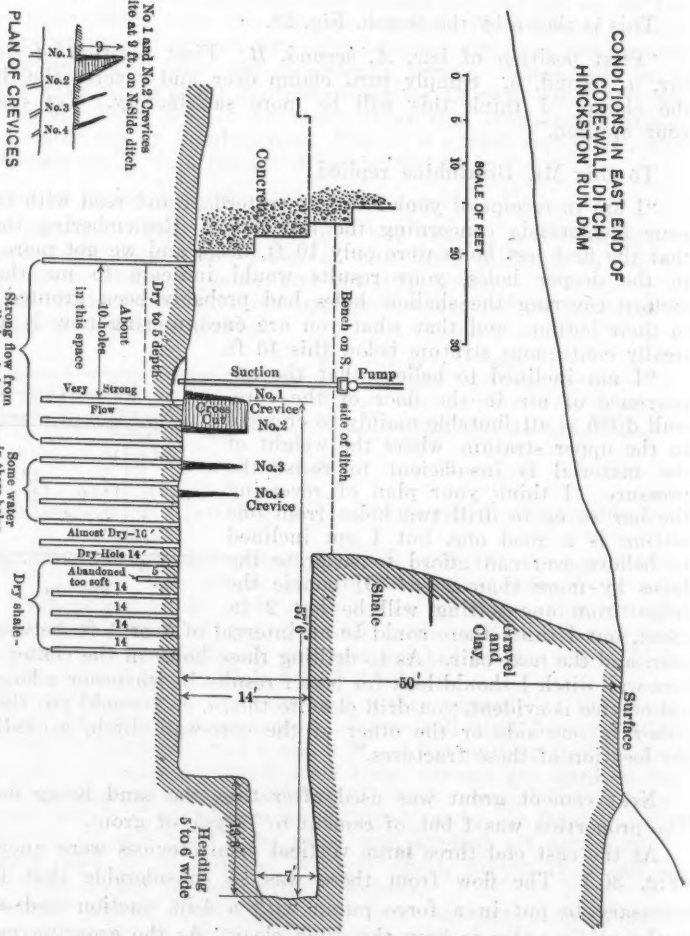
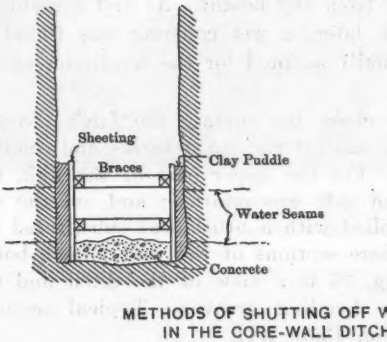
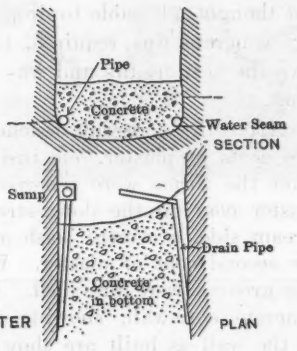


FIG. 30.

with trowels. Two coats of plaster were applied on the north or reservoir side and one on the south side. The suction pipe of the pump was built in concrete, and carried up with the wall. The strong flow of water in this section of the ditch made it difficult to place the concrete for the core-wall without having the cement washed out before it set. Mr. Souder.

At first, the method indicated in the sketch, Fig. 31, was tried, namely, a line of 1-in. sheeting was placed as shown, and clay was rammed between the sheeting and the rock to stop the water flow. Concrete was placed between the sheeting which was raised gradually as the concrete was carried up. This did not prove wholly satisfactory, and the method of piping the water directly to the sump was adopted, as shown in Fig. 32. This proved successful. Where there was too great a flow of water, plastering could not be done, but con-





METHODS OF SHUTTING OFF WATER IN THE CORE-WALL DITCH

FIG. 31. FIG. 32.

siderable neat cement was dumped in along the walls with the concrete and well worked in. After the concrete was well set, the pipes were plugged and the flow of water was shut off. As the concrete was carried up, the water came out of the ditch walls higher up, showing that the underflow had been intercepted. The core-wall ditch was extended well into the hills on each side of the valley, and test holes were bored for water, none being found, because, after a certain distance, the rock became hard and massive and free from seams. At the west end of the ditch, there was as much trouble with water. It would seem that the underflow of the whole valley was concentrated at this point, the rest of the ditch having been grouted and the flow cut off.

The drill-hole grouting method failed in the west side of the valley, as in the east, and here the bottom was also taken out, down to the water strata, and the water fought inch by inch by piping it

Mr. Souder.

from the streams to the sump, as before described, and the ditch was completely filled with concrete.

The proportion for the concrete in the core-wall was 1:2:5, generally; but, at the bottom, it was much richer in cement, which was not spared in efforts to make a tight job. Near the crevices a proportion of 1:1:2½ was used. These proportions had to be varied, also, to suit the sizes of the stone supplied, which varied from ½ in. at times to 3 in. The top section of the wall was made of 1:3:6 and 1:3:7 cinder concrete. The concrete was mixed in a machine of the continuous-mixer type consisting of a long square revolving box with a helix at the back. The machine was not wholly satisfactory, but was the best to be had. The concrete was received in ½-cu. yd. dump buckets, carried on small trucks running on light track to the derricks; it was then lowered to the bottom of the ditch and dumped, as it was not thought advisable to drop it from any height. At first a middling dry concrete was required, but, later, a wet concrete was found to give the best results and was finally adopted for the remainder of the work.

After the core-wall reached above the surface the faces received two coats of plaster, one inside against the wood forms and another after the forms were removed. For the upper part of the wall, the plaster coat on the down-stream side was omitted; and, on the up-stream side, a cement wash applied with a brush was substituted for the second coat of plaster. Where sections of the wall joined, bonding grooves were provided. Fig. 33 is a view of the ditch and the concrete core-wall, showing the bonding grooves. Typical sections of the wall as built are shown on Plate XII.

The concrete core-wall contained 10 840 cu. yd., and required 13 166 bbl. of cement, or an average of 1.21 bbl. per cu. yd. The grouting and plastering took 2 078 bbl. of cement, in addition. The exact quantity of cement used in grout alone is not known.

That the work was successful was shown by the breaking out of some small springs on the inner side of the reservoir at or about the elevation of the top of the core-wall.

The comparatively low pressure of 60 lb. was insisted upon, because it was thought that extreme pressures might do harm in opening up the seams unnecessarily, and thus cause blow-outs below the dam, such as have occurred, the writer is informed, in some recent applications of the grouting method.

The result obtained by all the precautions at Hinckston Run was, not only a water-tight and practically indestructible dam, but the people in Johnstown were completely reassured, thus permitting the company to proceed with the construction of other large reservoirs in the same district.

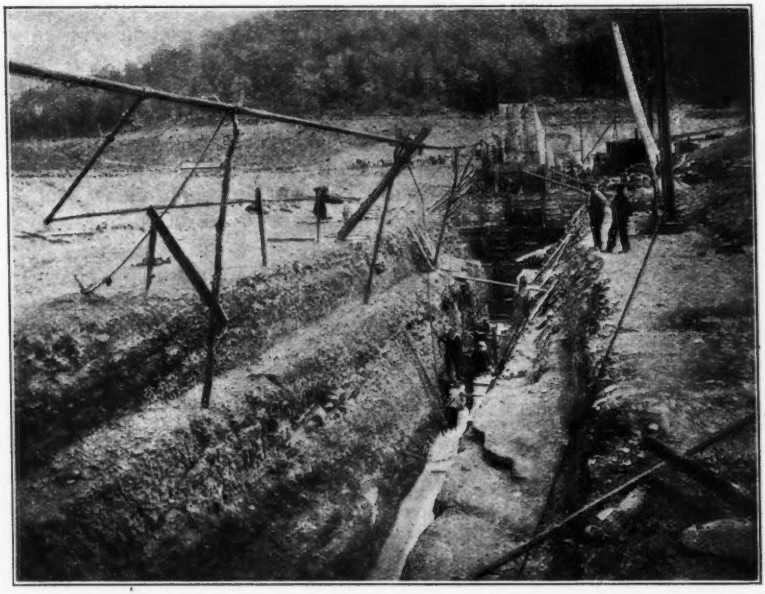


FIG. 33.—WESTERN SECTION OF CORE-WALL DITCH, HINCKSTON RUN DAM. PARTLY COMPLETED CORE-WALL, IN BACKGROUND, WITH VERTICAL BONDING GROOVES.

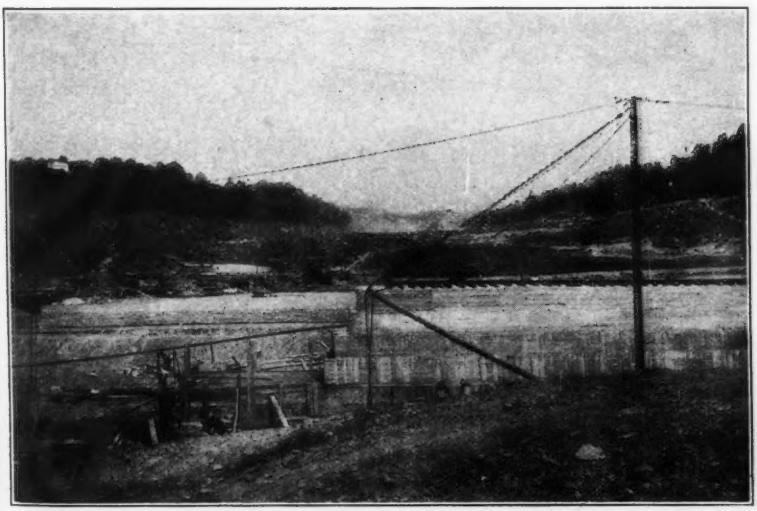
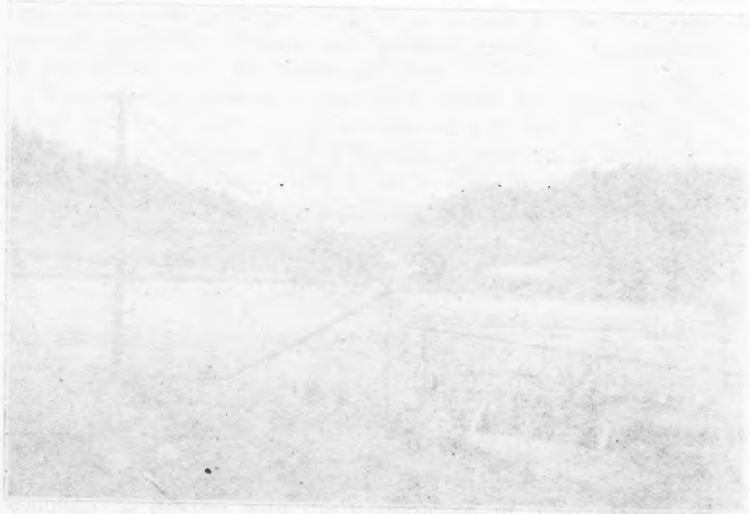


FIG. 34.—PARTLY COMPLETED CORE-WALL, HINCKSTON RUN DAM. LOOKING NORTH, TOWARD RESERVOIR.



Mr. Souder.

The writer considers the grouting for the Hinckston Run Dam to have been successful. In any case, extraordinary care must be taken in applying it, and then only after conscientious study of conditions, and actual tests to determine its availability for the given locality and material. The apparent success of the grouting method for cutting off the underflow at Estacada and other places would indicate careful attention to these requirements.

The writer questions the advisability of using air pressure much in excess of 60 lb. in work of this class.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.

Discussion.*

By A. H. Sabin, Assoc. M. Am. Soc. C. E.†

Mr. Sabin.

A. H. Sabin, Assoc. M. Am. Soc. C. E. (by letter).—It is not the desire of the writer to engage in unnecessary discussion, but it is desirable to correct misstatements of facts. Mr. Gardner questions the statements made as to the Havre de Grace Bridge tests. It is true that in the paper as first published the percentage of silicates, etc., in Paint No. 6 was too low, but a correction was made at the time of reading; the quantity of red lead was variously estimated at from 65 to 71% by different analysts. Paint No. 10, Mr. Gardner says, contained 10% of litharge, which was one analysis, but the same laboratory reported three other analyses of this paint showing 7.1, 6.8, and 8.5 per cent. Paint No. 11, he says, contained 5% of litharge; the four analyses give 3.51, 3.61, 2.99, and 4.14 per cent. The method used in the Contracts Laboratory‡ at that time gave results too low in red lead; the statement made by the writer was based on private information, and is probably accurate.

Mr. Gardner says:

"The only authoritative paint tests, to determine the efficiency of single pigments as preservers of iron and steel, which have ever been made, are those by the American Society for Testing Materials, at Atlantic City, N. J., in 1908."

If this is true, the writer should have mentioned it in the paper; it seems, therefore, incumbent on him to explain his views as to the matter. In the first place, he objects to the statement that these

*Continued from February, 1914, *Proceedings*.

† Author's closure.

‡ The analyses reported from the Contracts Laboratory may be found in *Proceedings*, Am. Soc. for Testing Materials, Vol. VIII, Plates I to III.

Mr. Sabin. tests were made by the American Society for Testing Materials. The plates were selected and cleaned, and the paints were designed, made, and applied by persons employed for the purpose by an association of manufacturers of mixed paints. The entire scheme and plan was theirs; and the only way in which that Society was mixed up in the matter was by accepting an invitation from the owners of the tests to inspect, by a committee, the prepared plates, as to the history of which they knew nothing except what the owners saw fit to tell them. According to Mr. Gardner, these tests show that no important single-pigment paint, and no simple mixture, such as can be made by an ordinary painter, gave results at all to be compared with the complex paints made by the people who designed the tests. Exception to this general statement may be taken in the case of a few pigments which are too costly to be used, especially the basic chromate of lead, which is described by Mr. Gardner as having "high litharge content", when in fact it contains no litharge at all.

The writer does not regard tests of this sort as authoritative or conclusive. He has before this been on a committee to inspect tests devised by Mr. Gardner, and has been impressed with the unfailing certainty with which they proved Mr. Gardner's contentions; and, for that reason, he can attach no value to them, however unprejudiced and competent may be the inspecting committee. This opinion is strengthened by the observations of A. W. Carpenter, M. Am. Soc. C. E., of the New York Central Railroad, and Mr. S. S. Voorhees of the Bureau of Standards.* Mr. Carpenter observed that in many cases three coats of paint on these panels were going to pieces in 20 months, though similar paints, according to his experience, would be better after an exposure two or three times as long, in New York City. Mr. Voorhees, who had been Chairman of the Committee on Protective Coatings from its beginning, said the Havre de Grace plates, after 5 years' exposure, were in better condition than those of the paint manufacturers' after 1½ years.

Mr. Toch's discussion contains two or three curious misstatements. He says that "every one of these red leads [on the Havre de Grace Bridge] was coated over with a carbon paint". According to the published analyses, as already referred to,† the pigments of Paints Nos. 10 and 11 showed only a trace of carbonaceous matter. He also says that his own paint was No. 12 and was the only one used in only two coats, all the others having three. The tables just referred to show complete analyses of each of three coats of paint on this section, the only one having but two being Section No. 1. It is reported‡ that:

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* *Proceedings*, Am. Soc. for Testing Materials, Vol. X, p. 90.

† Also Plate IV, Vol. VIII, *Proceedings*, Am. Soc. for Testing Materials.

‡ *Loc. cit.*, Vol. XIII, p. 338.

"Three paints (Nos. 6, 10 and 11) in Class I, under each of the separate spreading rates, may each well be designated as excellent. * * * What differentiates these paints from all others under observation is the fact that while all the other paints except one furnish their best protection, such as it is, under the 600 sq-ft. rate of spreading and are generally markedly less effective under the thinner film rate, these three show such slight variation under different rates of application as to appear equally protective under either." Mr. Sabin.

This disposes of Mr. Toch's statement that the report "describes as excellent nine paints, three of which are red lead". Mr. Toch also says: "every paint chemist knows that there are some red leads that contain nearly 1% of caustic soda"; as a matter of fact, all red lead is made in a reverberatory furnace, and it is plainly impossible for caustic soda to be present in such a product; if any were there, it would be quickly changed to carbonate; no one has ever seen any.

Mr. A. S. Cushman deplores the fact that the paper "entirely ignores the electrolytic theory of the mechanism of the reactions which lead to metallic corrosion." It may be that this criticism is justified; but the writer's view is that a discussion of the theory more properly belongs in a meeting of chemists rather than engineers. The writer believes that he has read everything Mr. Cushman has written on the subject which he plainly regards as peculiarly his own. The writer's ignorance, therefore, is not due to lack of instruction, doctrine, or reproof, but probably consists in a pernicious tendency to try to check up theory with practice, which, however useful in practical work, is sometimes greatly disapproved by the theorist. What Mr. Cushman regards as the true theory of the corrosion of iron is, in the writer's opinion, that the action of water alone is all that is necessary for such corrosion to go on, and that the accidental presence of anything else is needless and not to be considered. This theory involves the supposition that in all water there is a portion which is ionized; that is, part of the hydrogen exists uncombined with the rest of the molecule, an unstable condition, which results in the formation of new compounds when another substance, as iron, is present. This theory, as regards the corrosion of iron, was first proposed by Mr. W. R. Whitney, now Director of the Chemical Laboratory of the General Electric Company, several years before Mr. Cushman adopted it.* Mr. J. N. Friend, who is Principal of a technical college in England, has been the chief opponent of this theory; he has been supported by grants from the Carnegie Institute, of Washington, and has contributed papers to the *Proceedings* of the Iron and Steel Institute on the subject, in which he believes that he has experimentally disproved the theory in question. A full discussion may be

**Journal*, Am. Chemical Soc., 1903, pp. 394-406, Whitney; and *Proceedings*, Am. Soc. for Testing Materials, 1907, pp. 211-228, Cushman.

Mr. Sabin. found in Mr. Friend's book.* Mr. Whitney, in his original paper, pointed out the enormously greater theoretical efficiency of carbonic acid, which is probably not denied by any one; and as this acid is a normal ingredient in air and rain, and all natural waters, it has always appeared to the writer that the pure-water theory of corrosion was of doubtful practical value. Further, in view of the fact that no one doubts the existence of electrical disturbances coincident with chemical action in general, and especially as it has long been known that electric instability is provocative of corrosion of iron, it seems to be an arrogant assumption to claim the exclusive application of the term electrolytic to the very limited theory propounded by Mr. Whitney, a restriction never proposed by him.

The fact that iron does not rust in certain solutions was known many years ago. Such facts are explained, some of them, and perhaps all may be, by the modern theories of chemical action; and Mr. Cushman has laudably attempted to apply such knowledge to the paint industry. The question in regard to this is: are his inventions, whether patented or not, of any practical value? The only answer the writer can give is that they do not appear to be. Mr. Cushman seems to believe that a linseed oil film absorbs water from the air, that it is in a sense hygroscopic, and that part of the solid particles of pigment in a dried paint film are really in solution; that this water, which is to all intents in solution in the solidified oil, acts on the iron and causes corrosion, and if some slightly soluble chromate, for example, is in the pigment, it will be in solution in this water and will prevent or forbid, or, as he says, "inhibit" corrosion. It is a very pretty theory, but does it work?

In reply to this, attention is again directed to the official report.† After 5 years' exposure, nineteen test panels, representing as many paints, were removed from the Havre de Grace Bridge, photographed, and then the paint was thoroughly cleaned off and the surface of the metal inspected for rusting. Excepting a few spots, where the coating was obviously injured accidentally, the surface of most of these plates was in exactly as good condition as when first painted. According to Mr. Cushman's theory, all these plates should have rusted under the paint; none of the paints used contained what he regards as efficient inhibitors, and some of them contained large quantities of carbon, apparently lampblack, which Mr. Cushman says is the worst stimulator there is. The paint in all cases was thin, being three coats at a spreading rate of 900 sq. ft. per gal., or a total thickness of 0.005 in., and had been exposed to the moist salt air from Delaware Bay, at the mouth of the Susquehanna River, for 5 years. The writer thinks this proves that a good paint, properly used, keeps out

* "Corrosion of Iron and Steel," 1911, pp. 44-67.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. XI, pp. 175-180.

Mr. Sabin.

air and water; and, as long as it remains intact, it protects the metal; when it has deteriorated so much that it has holes through it, the air and water get to the iron, which begins to rust. As long as there are no holes, no inhibitor is needed; when there are holes, no inhibitor will do any good. No doubt there is a difference in the value of pigments; perhaps a soluble sulphate acts to some extent by conducting water through a film, and perhaps acts on the oil itself; perhaps a substance like carbonate of lime forms a soap with the oil and injures it in that way; but of more practical importance is the greater or less surface attraction between the pigments and the oil; in this respect the differences are great and characteristic. To illustrate this, let us remember that when one end of a small glass tube is put in water the water rises, as we say, by capillary action, in the tube; but, if mercury is substituted for water, the surface about the glass is depressed as though it were repelled. These are cases of surface tension and attraction. In an exactly similar way, some pigments have more or less surface attraction for oil. The most remarkable, and at the same time the most important, case is where, in the manufacture of white lead, the pigment is subjected to prolonged washing with pure water, then allowed to settle, and the wet pulp is agitated in a mixer with raw linseed oil. Although the oil is lighter than water, it displaces the water, mixes with the pigment, and forces out the water, which runs off the top, leaving the lead and oil almost free from moisture. Red lead may be treated in a somewhat similar manner, but of course it is not, as it is a furnace product. As far as the writer knows, no other pigment will act in this way; in most cases the presence of a little water in the pigment greatly obstructs its proper mixing with oil; but pigments differ greatly in this attraction for oil; which may partly account for the widely varying quantities of oil required by different pigments. To give an entirely different illustration: it is common knowledge among all paint makers and users that 1 quart of turpentine will thin a batch of paint as much as 2 quarts of linseed oil, because of the greater fluidity, or less viscosity, of the former. However, if we make a stiff paste with any ordinary pigment, as white lead, we require a certain quantity, say 8% of oil; if we substitute turpentine for half of this oil, it will take, not 2%, as might be thought, but 6 per cent. This remarkable fact leads to the consideration that the paste is a plastic body rather than a viscous one, and the function of the liquid is to stick the solid particles together, which the oil does much better than the turpentine, the surface attraction of which for the lead is very low; but, when more oil has been added, to make paint, which is a viscous but not a plastic body, turpentine is more efficient as a thinner than oil. In a similar way we account in part for the advantage of proper cleaning of steel before painting. It is well known that any ordinary oil easily

Mr. Sabin.

wets clean iron and steel, spreads over its surface, and is removed with difficulty; but, if the surface is fouled with anything which does not attract the oil, the latter when brushed out into a very thin film—and paint films are very thin—may break and leave holes, or pores, through which the atmospheric agencies get to the metal.

If, now, the pigment is one which has a great surface attraction for the oil, it is obvious that, when the paint is brushed out to a thin film, the presence of these solid particles in vast numbers will tend to hold the film together, to make it in fact tougher, so that it will be less likely to break into holes; and this is one reason why fineness is one of the most desirable qualities in a pigment; and why such a paint brushes out easily into a thin film, or, as we say, has good working quality. It is also a reason why a paint of good working quality (if it has no counterbalancing defects) is a good protective paint, for it makes a continuous film. Conversely, if the attraction between the pigment and the oil is slight, the paint is less likely to be satisfactory in any respect. Nor is this advantage likely to disappear when the film hardens; for the particles which attract the oil are likely to be more tightly cemented into the mass and form an element of strength rather than weakness; and if, in addition, the pigment is one which shows very little, or perhaps negative, attraction for water, it is likely to show great resistance to atmospheric action.

Linseed oil is an excellent insulator against the transmission of electric currents; and some pigments are themselves non-conductors, though others are the contrary. As it is generally agreed that electric tension is an effective cause of corrosion, it is desirable that paint for metal should contain pigments which will tend to insulate, rather than to break down the insulation given by the oil itself. This is a very different thing from Mr. Cushman's theory that certain pigments embedded in a cement of dried oil will act, in conjunction with pure water (which, in the writer's opinion, could not possibly get there), as a primary battery bringing the iron into solution. Mr. Cushman has elsewhere described the action of certain pigments as like a poison to iron; if correct, which in some cases at least may be doubted, it is because of their unfavorable relation to the oil, which renders them incapable of making good films, and the remedy should be looked for in this direction, rather than by his favorite prescription of a homeopathic dose of some inhibitor.

It is well known that some paints, including some varnish paints, and especially red lead in oil, adhere to iron and steel much better than others. Probably the mixture of a pigment in the vehicle affects its surface attraction to the metal. When we consider that often as much as one-third of the volume of a paint is composed of these solid particles, and that, in the case of a really fine pigment, there

Mr. Sabin.

are from 25 to 50 particles overlying one another in a film 0.002 in. in thickness (this supposes them to be 0.00001 in. in diameter, which is within the truth), it may be believed that the action of such a mixture is very different from that of oil alone.

Mr. Carpenter's discussion is interesting and valuable. In Table 1, showing his red lead experiments, it is notable that the quantity of paint used in the second series was 6.3% more than in the first (owing to the use of more pigment), but the surface covered was almost 30% more; and in three cases out of five the working quality of the paint was better, and equal in the other two. If a mixture of 30 lb. of red lead to 1 gal. of oil will work better and cover 28% more surface than a 24-lb. mixture, the fact is worth knowing, and should lead to considerable corrections in common practice.

The relative fineness in these tests was determined by the weight in grammes per cubic inch. The writer does not regard this as a safe method. Certainly, a very fine red lead, after being packed in a barrel in the usual way and then sifted to make it perfectly loose and open, will weigh several more grammes to the cubic inch than it did before being barreled; and it is probable that the volume depends as much on the friction between the particles as on anything, and this, again, may depend on their electrical condition, shape of particles (red lead particles differ in shape according to the material from which they are made, some of which is crystalline), and probably many other unknown causes. At all events, in the laboratory with which the writer is connected, the best-known volume apparatus gives results so widely and irregularly at variance with those of a more nearly absolute method which is checked by actual microscopic measurement, that it is not regarded by the writer as entitled to much consideration. It is possible that a fine red lead may contain considerable litharge, if it is taken out of the oxidizing furnace too soon; also, it is, at least theoretically, possible to oxidize highly a somewhat coarse material; but, in general, the finer the material the higher is the oxidation.

Mr. Carpenter's criticism of lack of definiteness as to the composition of the Havre de Grace tests has already been met by the figures given; attention is also asked to Mr. Wagner's discussion giving his experience with red lead ranging from 96.48 to 98.63% for 5 years, with satisfactory results. It should not be forgotten that, in the Havre de Grace tests, at least two of the red leads were what may fairly be called high grade, and were furnished by some of the largest and most experienced paint manufacturers in the country; and it is fair to infer that these makers desired to make the best possible record, and therefore that they had evidence, satisfactory to them, that this was the best material to use; as it proved to be.

Mr. Sabin.

As to why it should be better, the writer must admit that the reasons given in the paper are partly theoretical; but he does not see why he should be denied adopting a theory any more than other people, particularly as this one has nothing novel or original about it, is generally held by the trade, and has been forced on him by the constant experience of a quarter of a century. Age, however, does not make a theory right, any more than it does a man; but it entitles it to respectful consideration; it has successfully served a useful purpose for a long time. There is a broader ground, however, on which all can meet; for all agree that more than half the battle is in getting the paint applied to a clean surface in a smooth and uniform coating; smooth, because resistance to wear is much increased by the absence of ridges, lumps, and grains in the surface, and uniform, because thick masses of paint are waste, and thin places are weak. As Mr. Coombs puts it, a $5 man with a $1 paint is better than a $1 man with a $5 paint. Now the whole history of the improvement of red lead as a paint has been of this nature: to please the consumer, it has been made continually finer in texture, so as to be free from roughness and disposition to run, and lower in litharge, so as to be less active to oil and therefore more manageable. Tendency to settle is not so much a matter of specific gravity, but is caused by coarseness; no paint settles less than white zinc and white lead, which are very heavy but very fine; and no paint is more refractory in its general behavior than a low-grade, red-lead paint which has begun to get thick and viscid. It accords, then, with general experience, that a red lead which consists entirely of impalpably fine particles and has about the same relation to oil that white lead has, will be easily applied and form a smooth and uniform coat; and that, irrespective of the theoretical questions involved, it will practically be better on the average and last longer. This, it is submitted, is in accord with all experience with painting in general.

One more observation is suggested by Mr. Carpenter's discussion which is that it is difficult to determine spreading capacity in a panel test. The painter instinctively tries to spread all alike, so as to give each paint a fair chance; the more conscientious he is, the more his results may vary from what are actually secured in working under average conditions. It may be doubted whether a test of less than a barrel of paint is of much value in this regard.

In conclusion, the writer would explicitly deny the charge, made by Mr. Cushman and his associates of the paint manufacturers' organization, that he claims to have said the last word in respect to the protection of steel, and that there is no need of further investigation of paint problems. Such a charge will not be taken seriously by any member of the Society. Another assumption, that the paper was designed to create an interest in red lead, is really based on the fact that every one is interested in that substance, and when any one has

anything essentially new to say about it, whatever else he says is likely to be neglected. This very fact, that it is a widely known and valuable material, made it proper to mention, in a paper of this character, any important improvement in its manufacture or use, and the fact that the writer is able to speak authoritatively of its composition and character seemed a good reason for doing so. It is not a perfect paint; there is no paint suited for all uses; but that is no reason why we should not know as much as we can about any of them, particularly those which we use the most. Mr. Sabin.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINERS.*

By Carl H. Fuller, Assoc. M. Am. Soc. C. E.

Mr. Fuller.

Carl H. Fuller, Assoc. M. Am. Soc. C. E. (by letter).—The writer has been vitally interested in the report and the chart of the Special Committee which has been investigating the matter of compensation of engineers and in the discussion of the subject following its publication. The work of the Committee is certainly thorough and commendable, and, should its efforts result in improving the standards of the average member of the Profession, it will merit the gratitude of every member of the Society as well as the Profession at large.

However, the writer cannot help but feel that the chart fails to represent actual average conditions, owing to several reasons, one of which is the effect of the maximum compensation line, containing several of what may be called abnormally big incomes. It will be conceded that the *Transactions* of this Society, owing to the extremely technical nature of a large portion of the papers, do not represent the ideas of the majority, who have neither the time nor the qualifications to prepare and discuss the subjects presented, but, represent the opinions and results of the labors of a very small minority, who being favored with ability and opportunity, derive pleasure in devoting their time to research and such work. Thus, in the consideration of this subject, it appears that the discussions represent the academic opinions of those whose successes in business, if platted as an income line on this chart, would rise above the average, rather than the opinions of the majority, whose income line would fall materially below this average.

The object of every professional organization is the betterment of its individual members, and the matter of personal income is bound

* Continued from April, 1914, *Proceedings*.

Mr. Fuller.

to have its reflex influence on the purely professional or technical aspects of that society. From the very nature of things, every paper presented before this Society reflects the financial success of the writer, and there are without a doubt a large number of members, who, if it were not for the handicap of family and the necessity of providing for them, would be glad to contribute very materially to the *Transactions* of the Society. Therefore, if the Society cannot give serious consideration to the material, individual condition of its membership, it will fail to represent the ideals of the majority; thus, the matter of personal or professional income becomes as much a matter of concern to the Society collectively as it does individually.

Out of curiosity, the writer platted his own income line for 15 years, superimposed it on the chart, and found that, being a very ragged line, it projected alternately above and sometimes very considerably below the average line on the chart, although a modified line of averages would follow the average on the chart quite closely. Nor does the writer believe that his success is materially different from that of the majority of engineers with whom he is acquainted, and the conclusion is reached from experience and observation that this Profession is one of constant change and vicissitude, as few engineering positions can reasonably be expected to be permanent. In no way can the Profession be compared with any other professions or industries, and the completion of a building, a railroad, or other project, naturally releases one or many engineers from an engagement that may perhaps reflect the greatest credit on their ability. Then again, the public in general has a very vague conception of an engineer's work, which, in a measure, is due to its transitory nature; consequently, many irregular employers of engineering services fail to appreciate the attainments of a man qualified to perform this kind of work and incidentally appraise his services at the valuation in dollars and cents that he himself places on them.

Architects, in their societies, have found it no disgrace to define their compensation, and physicians and those of other professional organizations have scheduled the value of their services in varying degrees; therefore, why should not this Society endeavor to establish a higher standard, which will command the respect of other men. There is no doubt that the compensation for engineers is less than that for men in other lines of industry, and as the public very largely measures a man's success and ability by the amount of compensation he is able to demand for his services, it behooves the Society and the Profession to protect its professional honor by demanding for its members fees which are consistent with the services rendered.

Conservation and scientific management have become quite a fetish in the Engineering Profession, yet, as a whole, it is more extravagant and wasteful of its energies than any other body of intelligent labor

in the world. If this statement is challenged, one item alone will be considered: The average engineering job (not position) lasts from 3 to 18 months, occasionally longer. In the transition from one job to another the average engineer will lose at least a month, and, if his last project has been finished late in the year, he is likely to lose 3 or 4 months. Without resorting to statistics, the writer thinks it may safely be assumed that the average engineer loses on this account fully one-tenth of his time and income, through no fault of his. If this waste can be eliminated—for waste it is—not only will the average income line on the chart be raised, but a very material difference will be made in the minimum line, for this waste naturally applies to those whose incomes fall in the lower half of the chart, and among whom there are many able men. Is not this waste as worthy an object of conservation for the Society to consider as that of the water-powers and forests? Mr. Fuller.

Some one has remarked that a capable engineer will always have a job, but this statement needs to be examined. Is it the man who is capable from an engineering standpoint, or the man who is a good salesman of his services that gets the job? The writer is inclined to think that it is the latter; for, getting a job requires salesmanship of the highest degree, more so than selling rails or cement. Consider the men who maintain large offices successfully. Without any discredit to them, let it be asked how much work they get because of their engineering ability, and how much they get from salesmanship? The latter often brings the larger fees. The writer knows an engineer, whose name has been connected with monumental work all over the United States, who cannot perform the simplest equations, yet, from his sales ability and knowledge of men, he has a large practice. If the writer should mention his name he would be accused of slander. Therefore the statement that, if a man has ability he will always have a job, is fallacious.

From these thoughts arise two ideas: (*a*) Can the Society as a body wield an influence that will tend to increase the compensation of its members and the Profession at large so that it will be consistent with services rendered? (*b*) Can it do anything to prevent the loss of time of its members? The writer thinks it can, and, furthermore, that it is one of the obligations of its existence, because no other body of men is qualified to do this.

There are many phases to this proposition. It is the biggest problem that has ever been presented to the Society. Is the Society going to table it, after some academic discussion, or is it going to seek a remedy? The writer would suggest that a standing committee be appointed to continue the work that has been started.

As to the conservation of the time lost, the Society can, if it will, afford immediate and effective results by the establishment of

Mr. Fuller.

a secretaryship or bureau, under the direction of a competent engineer who knows the work and the men, the duties of which will be to furnish a clearing house for the services of its members. In this the employing member will receive as much benefit as he who is seeking employment, for the reason that a properly conducted department of this kind will try to place the right man in the right place, instead of "bleeding" an unemployed man who can ill afford it. The writer is of the opinion that every member who has used the employment agency to secure help has been disappointed in the results obtained, because the employment agent is only looking for his commission. In one instance an agent offered to divide his commission if the writer would employ his clients and discharge them again when the fee had been paid.

Some member on whom fortune has smiled, in the way of successful positions, may object that it lowers the dignity of the Society to go into the employment agency business; but, who has a greater right to conserve the time and compensation of its membership than this honorable body. Each and every member belongs to this Society for one of two reasons: either to derive some benefit from the organization or to confer some good on his fellow-members. Therefore, while the Society is caring for the intellectual welfare of its members, why not go a step farther, and provide for their material welfare, thereby permitting them to derive greater intellectual benefit. The writer hopes that there will be further discussion on this subject from the majority of the membership whose income falls below the average line, in order that some light may be obtained on the whys and wherefores.

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STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

BY MESSRS. E. P. GOODRICH AND JAMES L. TIGHE.

Mr. Goodrich.

E. P. GOODRICH, M. AM. SOC. C. E. (by letter).—What Mr. Hazen has done in the design of probability paper is unique, and, with easily obtainable information as to the meaning of lines drawn on it in relation to probability formulas, his new device should be of almost as much value to engineers as the well-known logarithmic paper.

The writer believes that the work Mr. Hazen has done in applying the theory of probabilities to the data on annual stream flow and required storage, is a material advance in engineering science. Mr. Hazen has shown conclusively, in the writer's opinion, that the methods of computation to be followed in utilizing the theories of probability are applicable to water problems. The next step in the solution must be what is normally called that of inverse probability or the breaking up of the probability curve found from actual data in any case (such as that prepared by Mr. Hazen and shown in Fig. 29) into several possible component probability curves. This work is exactly comparable with the analysis which is now so common with reference to tidal and similar observations, which have been found to follow compound harmonic relations, breaking up this combination curve into the several simple harmonic ones which, when recombined, will produce the actual result observed in Nature. In the case of the harmonic curves, the tracing back of the several simple harmonic functions to their causes has been found relatively simple. It is similarly to be expected that, if Mr. Hazen's combination probability curve can be analyzed into two or more simple normal probability curves, in combination with one or more other simpler curves, the causes operating

* Continued from March, 1914, *Proceedings*.

Mr. Goodrich.

to produce the latter can be discovered, and further refinements can be worked out in the engineering of water-works design.

Several explanations of the skew curve, which Mr. Hazen found with regard to run-off data, are possible. One such explanation, which seems to the writer more than possible is that a normal probability variation is superposed upon a condition which would produce a periodic variation of long range. Evidently, if the data happened to have been secured for years which range near the minimum of a periodic fluctuation, the exact conditions, with reference to the data found by Mr. Hazen, would be reproduced. In the paper Mr. Hazen makes reference to such periodic variations in rainfall and run-off, and it is the writer's idea that the skew nature of the curve found by him indicates, primarily, that the data do not cover a large enough range of years to be entirely conclusive in the deductions which can be drawn. In the meantime Mr. Hazen's work will represent the acme to date.

Mr. Tighe.

JAMES L. TIGHE, M. AM. SOC. C. E. (by letter).—The ingenious and unique methods of analysis followed in the production of this remarkable paper, and the conclusions and results therein set forth, must be of the greatest interest to all hydraulic engineers.

In the earlier days of providing domestic water supplies from catchment areas, little attention was given to the relation between run-off and storage, with the result, naturally, that deficiencies generally occurred in the drier years. After a time these deficiencies led to the establishment of empirical rules or formulas for storage based on low rainfalls, or, where run-off records were available, to the tentative and laborious methods of tabulation.

It was not until the early Eighties that W. Rippl, Docent at the Royal Technical High School at Gratz (Styria), made public his graphical method for the calculation of storage known as the mass-curve method. As originally given, this method was followed in such calculations until the report on the water supply of New York by John R. Freeman, M. Am. Soc. C. E., was published in 1900. In this report a more convenient mass-curve method was shown, in which a single curve served for all rates of draft, whereas, in the original Rippl method, a new curve had to be made for each rate.

In this mass-curve method and in the other methods for calculating storage, it was generally sought to calculate from the rainfall or stream-flow records of the particular catchment area in question, or, if these were not available, from the rainfall or stream-flow records of some other catchment area applicable, the storage required to maintain certain drafts through the driest period covered by the records, and little if any attention was given to other important phases of the question, outside of evaporation, until the presentation of Mr. Hazen's paper.

Mr. Tighe.

In this exhaustive analysis, some new ideas relative to storage, which were never brought into the question before, have been treated at great length. Of these, the amplification of the dry years and their recurrences, the normal storage diagram, and the coefficient of variation may be mentioned. For the latter, hydraulics is under great obligations to the author, inasmuch as it must be admitted that, up to the present time, the science gave no yardstick whatever by which the variation in stream flow could be determined.

The coefficient of variation proposed is an ingenious one, and will not only serve the purpose for which it is used in the paper, that is, for storage calculation, but will enable a comparison to be made of the variation of streams, not only in one section or territory, but all over the world.

As the variation of a stream, however, is influenced, among other things, by the area of its catchment, inasmuch as the greater this area, the less variation the stream is likely to have, perhaps the author at some future time could expand this idea to cover the catchment area. This might not be included in terms of the coefficient, but possibly in some other way so that when comparing two or any number of streams, relative to variation, their catchment areas would be brought into the comparison.

In regard to the normal storage diagram developed, this, if the writer understands it correctly, is not based on the actual records of the flow of any one particular stream, but on an artificial record formed from the records of the flow of all the streams used in the analysis. Such an artificial record makes a series covering a period of 300 years, or thereabouts, and may be open to the objection suggested by the author, that it does not reflect any climatic changes which might occur in the actual series, were it not for the fact that such changes or movements are on such a large scale of time that any objection of this kind may be disregarded.

The utility of the normal storage diagram is shown by the fact that by this method the storage required on any stream, to maintain a certain draft, can be determined more accurately than from the actual run-off records of the stream itself, unless perhaps these records covered an exceedingly long term of years, many times longer than the longest now in existence. This diagram method is a great advancement in storage calculation, and very likely will be used as the standard method in such calculations in the future. Even were it the case that actual run-off records could be depended on for storage calculation, this normal storage diagram would be valuable as a check.

A very important point stated by the author is that the storage on a tributary, within certain limits is as useful as that on the main stream. This no doubt is true within the limits stated in respect to reasonably large storage, but is it true in respect to small storage

Mr. Tighe.

where the capacity of the reservoir and the draft thereon would be such that the reservoir would empty and fill several times a year or several times in the summer season? For instance, if the capacity of the Borden Brook Reservoir, of the City of Springfield, referred to by Mr. Hazen, was only 70 000 000 gal., would this reservoir, with its 8 sq. miles of catchment, be as efficient as the intake reservoir of the same capacity with its 48 sq. miles of catchment, assuming that the pipe line had a discharging capacity of 20 000 000 gal. per day?

The writer has made no test figures on this, yet it seems to him that the intake reservoir would be the more efficient, as it would be possible for it to fill more often in the summer season, because of its larger catchment, than the Borden Brook Reservoir.

Another point that should not be overlooked, relative to storage on tributaries, is the water that is lost by percolation and evaporation in its course from the storage to the main stream and intake, especially if conveyed intermittently in the old stream bed.

Although considerable attention has been given to the loss of water by evaporation from the surface of reservoirs, it seems that much less has been given to the loss of water by percolation from reservoirs.

When a reservoir is built and filled, the level of the ground-water above the dam is raised and, consequently, some extra storage capacity is provided beyond the apparent capacity of the reservoir, especially if the ground surrounding the reservoir is porous.

On the other hand, because of the ground-water being raised above the dam, if there is any connection between this and the ground-water below the dam, the flow of the latter, owing to the higher head, must be increased, and this increase will come or be fed from the reservoir. It can be argued, of course, that, with the foundations of a dam resting in impermeable material, percolation from the reservoir, under the dam at least, might be considered negligible. The writer would like to get the views of the author and of others on this matter, as he thinks that the loss of water from the average reservoir through percolation is more of a factor than is supposed. If this is the case, it seems that, in storage calculation, especially as other phases of the question are being so closely analyzed, percolation should also be considered.

In conclusion, the writer, under whose supervision the daily flow of the Manhan River, furnished the author, was measured, and who naturally has been interested in and has given some attention to the matter of stream flow and storage, considers this paper the most comprehensive analysis that has ever been presented on the subject. The author certainly deserves the thanks of all hydraulic engineers for the exhaustive manner in which he has treated the subject and the new light which he has thrown on it, especially those phases of it which hitherto have been neglected.

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AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

Discussion.*

By James Owen, M. Am. Soc. C. E.

James Owen, M. Am. Soc. C. E.—This question of sand-clay roads was initiated about 10 years ago at the suggestion of the United States Office of Public Roads, and the writer believes that roads of that type were afterward constructed in South Carolina. The idea was suggested to the writer by his experience in the construction of a speedway, for the surface of which no natural materials were available, so that they had to be manufactured in the best possible manner, and the mixture that had been made in South Carolina was tried. There was available some very good, greasy, gray clay, which was essential for the work, and about 2 miles away there was a very good bed of sand to mix with it. The writer, however, made five or six mixtures of the sand and clay before he obtained what might be considered a perfectly consolidated surface. These mixtures were put down in places where there was considerable travel, and thus the best method of surfacing was ascertained. Mr. Owen.

The speedway was built with the mixture giving the best results; the clay was spread over it in lumps; the sand was then put on top of that, and the two were spaded together and rolled.

One interesting thing happened in these experiments. The mixture was made according to a preconceived idea of the standard surface, and when it was completed it was impossible to get a smooth trotting way for horses. The surface was rolled, but it would not come down, under the conditions prevailing at that time. By accident, a bed of what is known as New Jersey loam was found, and a coating of this, from ¼ to ½ in. thick, was put on, rolled down, and,

* Continued from April, 1914, *Proceedings*.

Mr. Owen.

with the consolidation of the gravel and the clay on top, and the addition of the loam, a perfectly smooth and constant roadway surface was obtained.

It is well to note here that, in the construction of sand-clay roads, strange results may be obtained. In New Jersey there are two classifications of roads; in the north they are of stone, in the south, gravel. In both sections there is an enormous amount of travel, automobile and horse, and the experience of to-day tends to confirm the belief that the gravel roads are much more constant in uniform maintenance, much cheaper to repair, and much more satisfactory to travel on than the stone roads, and here comes in the whole question, as enunciated in this paper, that the original idea of macadam roads as the best means of travel under the old regime, of horse travel and hard steel tire travel, has passed away; that in all localities of the United States there is material at hand which should be used for the purpose for which it is required, and manipulated so that the results are good.

Roadmakers in the future must not rely on stone roads to maintain the desired conditions for the wear and tear of travel.

In the case of the Georgia roads, the climate must be considered, and this may be more serious than is thought at first. There the frost does not penetrate to any great depth; there is rarely any frost, except in the high lands; but that does not affect the maintenance of roads, and, as far as that is concerned, the practice that is applicable to Georgia may not be applicable in the Northern States, where frost often penetrates the ground from 2 to 3 ft.

Experience to-day in New Jersey, however, shows that there is no doubt that it is much more economical to use the natural material, properly manipulated, than to import ideal material to make the ideal road, which in the end does not give ideal results.

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CINDER CONCRETE FLOORS.

Discussion.*

By Messrs. Arthur H. Diamant and J. R. Worcester.

Mr. Diamant.

Arthur H. Diamant, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest, and, having had experience during the past 5 years in the use of reinforced cinder concrete, agrees with the author that the present method of tests of floor slabs does not conform to conditions on construction work.

Aside from the design, however, the great fault in the construction of such floors is the lax supervision in placing the material. The majority of apartment and loft buildings in New York City are erected by speculators. The men in charge of the erection of these structures for the builders know very little about reinforced concrete work, and are rarely on the floors where the arches are being placed. The inspector of the Building Department has so much work to supervise, that he can devote very little time to this part of his duties, and the result is that there is too much dependence on the contractor to do good work. His men, usually unskilled laborers, are required to fill a certain number of centers in a specified time, and, in order to do this, sufficient care is not taken to place the reinforcing metal properly. With the exception of wire-cloth, the spacing of the metal is usually guessed at, and thus its weight per square foot of arch is often insufficient.

Another cause of trouble is in the mixture of the cinder concrete. When hand mixing is done, there is very little attempt to measure the ingredients, and, in machine mixing, the cement, sand, and cinder hoppers are not kept filled. The writer was on the tenth floor of a hotel, being erected under his supervision, when he noticed that the cinder

*This discussion (of the paper by Guy B. Waite, M. Am. Soc. C. E., published in April, 1914, *Proceedings*, and presented at the meeting of May 6th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Diamant. concrete which was being hoisted did not seem to have the proper proportion of cement. The contractor had an efficient mixer plant in the cellar, and when the cause of the poor mix was investigated, it was found that the laborers were so busy keeping the sand and cinder bins filled that they neglected the cement bin.

One of the most pernicious faults is the cutting of holes in concrete arches shortly after they are laid. In an inspection of a large loft building, the writer condemned a panel of reinforced cinder concrete floor which adjoined a stair-well. A large hole had been cut through this panel for a 5-in. soil pipe, and smaller holes for the passage of water, steam, and electric pipes. The reinforcing rods had been cut and bent so that they would not interfere with the pipes. Before the arch could be taken down, it failed because a barrel of lime was rolled over it. There is no excuse for work of that kind, for, in a properly designed structure, the location of all pipe lines is known, and openings can be left in the arches when they are poured.

In conclusion, the writer wishes to state once more that, even though the floor slab is well designed, the careful work of the engineer is not properly carried out because of inefficient supervision in the field.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E. (by letter).—The argument of the author in favor of establishing allowable unit stresses and physical properties of cinder concrete as a means of calculating safe loads for certain designs is sound and convincing. Though there are some difficulties in the way of reaching satisfactory units, they should be overcome if we consider that the margin of inaccuracy will be much less than in any attempt to determine strength by crude, isolated, field tests of actual construction.

The chief uncertainty with regard to the material is on account of the wide range in quality of cinders, running, as they do, all the way from a soft, impalpable ash to a vitrified clinker. The writer has not seen a satisfactory specification for cinders, by which good material for concrete can be distinguished from bad. It is easy enough to write one which will exclude bad material, but the trouble with it would be that it would be so stringent that no material could be obtained which would comply with it. It is to be hoped that further light will be shed on this subject. Another uncertainty in the value of the best cinders lies in the great irregularity in the proportion of fine particles, corresponding to sand, which is contained in every load. This variation is so great in practice that frequently a batch of concrete may contain scarcely any material which could be classed as coarse aggregate. Under such circumstances we would obtain for a product, using 2 parts of sand and 5 of cinders, a 1:7 mortar, of which part of the aggregate is less hard than sand. It is chiefly to these peculiarities that we may attribute the wide discrepancies in the results of experi-

ments with cinder concrete, as shown in Table 3, and the general disrepute which the material has among structural engineers. Mr. Worcester.

It is true that cinder concrete slabs, in spite of their disadvantages, have been used to a phenomenal extent, and it is also a fact that failures which can be shown to be due to the use of this material are remarkably few in number. This being the case, it certainly behooves the Profession to deal with the subject in a tolerant spirit, and to do what it can to lay down rules for its safe use.

This Society's Special Committee on Concrete and Reinforced Concrete, in its report presented in 1913 prescribed units, which, though tentative, seem to correspond fairly well with present practice. The subject of the modulus of elasticity of cinder concrete is not referred to in that report, presumably being classed with concretes having an ultimate strength of less than 2 200 lb. per sq. in., for which the ratio of moduli is to be taken as 15. This low value of the ratio does not correspond with general practice. With this exception, the rules laid down in the report cover the method of computation and manipulation quite comprehensively.

Although cinder concrete may prove entirely satisfactory in floor and roof slabs, it is altogether too uncertain a material to recommend for beams, girders, and other structural members, and it is objectionable even for slabs when the span is too great. The reason for this seems to be that the softness of the aggregate allows an inelastic set, which becomes apparent in the deflection when the span is sufficient. It may not be possible to make a hard and fast rule limiting the length of span, for in some places the deflection may not be of importance, or, it might be compensated by cambering the slabs, but the peculiarity should not be lost sight of.

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REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.

Discussion.*

By Edward Flad, M. Am. Soc. C. E.†

Mr. Flad.

Edward Flad, M. Am. Soc. C. E. (by letter).—Mr. Finch suggests that the working tensile strength allowed for the concrete, 290 lb. per sq. in., is excessive. He is correct in his statement that Mr. Andrews advises the use of this stress for 1:1:2 concrete. The writer is unable to state definitely at this date the line of reasoning that led to the selection of 290 lb. as a safe stress. Probably the original intention was to use 1:1:2 concrete, allowing the stress recommended by Mr. Andrews, and, later, when it was decided to use a 1:1½:3 mixture, it was concluded that the stress of 290 lb. would be well within the probable ultimate strength of that concrete; moreover an additional factor of safety was provided in assuming a high value for the modulus of elasticity of the concrete. In the structure as built no vertical cracks have been observed, but there is still evidence of moisture at some of the horizontal joints, due probably to a failure to clean the old surface properly before depositing the new concrete.

The construction suggested by Mr. Finch and Mr. Potter for the joint between the side and bottom of the tank appeals to the writer as an improvement, except for the difficulty of making this joint watertight. In the design of the St. Louis Reservoir, the stresses at this point could only be determined approximately, there being a combination of cantilever action and hoop tension, and, in this respect also, because of the extra expense involved, the design is inferior to one containing an expansion joint at the junction of the side with the bottom.

* Continued from March, 1914, *Proceedings*.
† Author's closure.

Mr. Flad.

The arrangement for the removal of sludge while in operation, suggested by Mr. Potter, would have been of doubtful benefit for this particular reservoir, as there are two preliminary steel settling tanks, each 75 ft. in diameter and 25 ft. high, through which the water passes and from which the greater quantity of the sludge is removed before the water enters the concrete reservoir; there is no difficulty about arranging for cleaning the reservoir at times when it will not interfere with the operation of the plant, and the proportion of water to sludge required in cleaning is probably less than would be required with the plan of removal during continuous operation.

It is gratifying to note that Mr. Buerger's interesting comparison of designs containing cantilever walls with those providing for tension rings leads to the conclusion that the design selected, to wit, the tension-ring type, was practically justified, his theoretical determination of economical diameter for the reservoir in question being 134 ft., whereas the actual diameter is 150 ft. As Mr. Buerger surmises, the nature of the foundation available was really the controlling feature in deciding between the two types of reservoirs. A uniform distribution of the load was considered to be of paramount importance.

Mr. Buel seems to be in error in stating that, according to the formula given for the thickness of the concrete wall, the ratio of stress in the steel to stress in the concrete is 9 to 1. The formula is supposed to provide a ratio of 10 to 1, and is derived as follows:

Designating quantities and dimensions by the letters used in the paper:

Area of concrete per vertical foot of wall $+ 12\ T - A$,

Hoop tension $= pr = A\ s + (12\ T - A)c$,

$$pr = A\ s + 12\ Tc - A\ c,$$

$$T = \frac{pr - A\ (s - c)}{12\ c},$$

Assuming that $s = 10c$,

$$T = \frac{pr - 9\ Ac}{12\ c},$$

which is the formula given in the paper.

The description by Mr. Wegmann of the construction and failure of the Stony River Dam emphasizes again the desirability of carrying cut-off walls down to rock or impervious strata. Happily, the failure was not as serious as at first reported, but it furnished a tender morsel for those who, through personal interest, were unfriendly to the Ambursen Company.

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THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

Discussion.*

By Messrs. M. J. Butler, Albert Lucius, Henry W. Hodge, Charles Evan Fowler, L. J. Le Conte, N. Petinot, Leon S. Moisseiff, and F. W. Skinner.

Mr. Butler.

M. J. Butler, M. Am. Soc. C. E. (by letter).—The author has once more placed the members of the Society under obligations. He has dealt most ably with a matter of great interest and importance.

It is an error to assume that the limit has been reached with ordinary carbon steel—undoubtedly, if the customer is willing to pay the price, the steel maker will supply a much better quality of steel:

(*a*) By fluid compression of the ingots;
(*b*) By careful heat treatment and lighter and slower blooming, putting more work on the bloom;
(*c*) Cropping away all segregated, piped, and inferior metal;
(*d*) Careful re-heating of billets, slower rolling, and better work generally on the rolls and sections;
(*e*) Such a quality of acid steels—possibly with a cheaper alloy than nickel—would allow of higher unit stress in large members, where shock and impact would be amply cared for.

Alloy steel of nickel and other materials may be secured to meet any reasonable requirements. Steel makers will rise to the occasion, when the demand justifies the outlay in getting ready for steel of such a quality.

In very great bridges, such as are under consideration in this paper, the increased price of the metal would not be so serious; there are too

*This discussion (of the paper by J. A. L. Waddell, M. Am. Soc. C. E., published in March, 1914, *Proceedings* and presented at the meeting of April 15th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Butler. many other factors that go to make the cost, such as the substructure, the erection of great cantilevers, and the special tools, plant, and shop, to fabricate the member. The engineering and the financial costs are to a considerable extent independent.

Mr. Lucius. ALBERT LUCIUS, M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest. Its objects are stated to be: First, to give the weights of different span lengths for simple spans and cantilever bridges, using metals of various elastic limits; secondly, to indicate the extreme practical limits of span lengths of cantilever bridges constructed of such metals; and, thirdly, to demonstrate the comparative economics of finished bridges involved in using metals of the various assumed elastic limits.

Concerning the weights of long-span bridges built of assumed metals of higher elastic limits, whatever their actual amount may be, there is little doubt that weights would reduce about as formulated in the paper if the unit stresses follow the elastic limits, as they should, provided the metals have the same superior properties as to ductility which they have as to strength, for then only would confidence in the material be justified to the extent of developing all sections and all details carrying calculated stress by the proportional unit strains. Members and their connections and all details, the function of which is to distribute local strains over sectional areas, could then be made of the same material working under the same stress and elastic condition, but, if the high alloys are of inferior ductility and connections and distributing details must be made of softer and weaker metals, and the unit amounts of extension and compression vary in the different parts forming the ends of a member, the difficulties of making proper connections would be increased and the advantages of reduced main sections would be partly neutralized.

Concerning the extreme length of cantilever spans, it is the writer's opinion that it has been reached, as far as practicable; that it will not be carried very much further, even if higher grade material is available; and that every consideration of reliability to carry load, simplicity of the carrying member, availability of the highest grade of material, and ease of construction and erection, points to the suspension bridge as the type to take care of extreme span lengths where bridges are preferred to subway constructions.

Concerning the economics involved in the use of high elastic limit materials, these are expressed most readily in a sufficiently satisfactory way by the ratio of elastic limit stress to price per pound of the metal, and whichever metal sells at the least price for the highest elastic limit, all other properties being in proper relation to it, is the cheaper material, and this would apply to the various types of bridges alike as far as their fabricated cost is concerned. Generally,

the idea of using simple spans up to their extreme limit of possible application cannot be attractive to engineers, because there is hardly any other reason for long spans than the requirement to maintain an unobstructed right of way under these proposed spans. The feasibility and cost of erection will determine the type, and a cantilever or continuous span over several openings, or an arch bridge, all of which can be erected cantilever fashion, would probably be selected before the economic limit of the fabricated simple-span bridge is reached. Besides, in long-span bridges, especially where prominently placed, esthetic considerations do and should enter more emphatically and obscure the importance of relative economics still further. Mr. Lucius.

Concerning the desirability of improving on the present available bridge material, both as to uniformity of present properties and in regard to increase of all its elastic properties and its ductility, there is probably no difference of opinion among engineers, and each, no doubt, does all he can to encourage and help push manufacturers to favorable solutions; but this is another subject, and in the care of the metallurgist. It is hardly likely that it is inertia alone which makes progress slow and apparently does no more than maintain a stationary level. It is more likely that there is a greater possibility of making economic improvements on the present grade of steel by cheapening it and making it more uniform in quality, than to make a higher grade of steel at a price which will give it an advantage commercially, except for special purposes, in which case the price is not so important relatively to the whole special purpose to be accomplished. The vast majority of railroad bridges can be well taken care of with the present grade of steel, and whether lighter sections of higher grade steel worked to higher strains would be an all-round improvement for the bulk of bridgework, might be open to debate. It certainly would not be the case, in the writer's judgment, if higher strength did not also carry with it higher ductility.

The engineer must get next to the metallurgist. It is highly probable that any metallurgist who knows how to produce a better steel alloy at commercial value will succeed very promptly in getting it on the market, and engineers would accept it as promptly and adjust their constructions to it.

HENRY W. HODGE, M. AM. SOC. C. E.—The length of bridge spans in general use has been increasing steadily, and we have reached limits where the dead weight of the structure has become the largest portion of its carrying capacity, so that some method of keeping the weight down is a necessity for the construction of the great spans now contemplated. The only way to reduce the dead load materially is by the use of metal of higher carrying capacity than our present materials, and a long step in this direction has been made by the use of nickel Mr. Hodge.

Mr. Hodge. steel, which has 50% greater carrying capacity than the carbon steel in general use.

The trusses of the three 668-ft. spans of the St. Louis Bridge were designed for nickel steel throughout, except certain minor sub-members. Nickel-steel eye-bars and carbon-steel compression members were also used, the floor system and bracing being of carbon steel in each case.

The weights of each span were:

With complete nickel-steel trusses..... 9 200 000 lb.
With nickel-steel bars, and the rest of carbon steel .....10 900 000 lb.

The dead load of railways, tracks, etc., was 5 500 lb. per lin. ft., so that the total average dead load was:

With nickel-steel trusses.....19 300 lb. per lin. ft.
With nickel-steel bars, and the rest of carbon steel .....21 800 lb. per lin. ft.

Thus, the nickel-steel compression members in the trusses made a difference in weight of 2 500 lb. per lin. ft., or 13 per cent.

The average live load on the two decks was equal to 16 600 lb. per lin. ft., thus the use of nickel-steel compression members made a saving of 7% in the total load on the structure.

The average unit prices for the two classes of material in this structure, erected in place, were:

Nickel steel, 5.6 cents per lb.
Carbon steel, 3.95 " " "

making a difference of 1.65 cents, or 42% of the price of the carbon steel; but the elastic limit required for the nickel steel was 50% higher than for the carbon steel, so that the nickel steel was the cheaper, strength for strength.

This steel had 3¼% of nickel, but manufacturers are now commercially producing an alloy steel, with not more than 1½% of nickel, together with small percentages of chromium and vanadium, which has all the properties of this steel, at a very much reduced price, so that there is at present a readily obtainable material, which is 50% stronger than the carbon steel in general use, at a comparatively small increase in cost.

This increase of elastic limit to 50 000 or 60 000 lb. per sq. in. will help greatly in the construction of spans of considerable length; but, for the very long spans now being planned, a still stronger material is needed, and can economically be used at a very considerable increase in price. In the design for the 2 880-ft. suspension span for the North River Bridge at New York, there is great economy in placing the stiffening trusses along the cables, so that articulated

joints are required, thus necessitating eye-bars in place of wire, which has heretofore been used in long suspension bridges. Mr. Hodge.

The total live load on the eight tracks, two roadways, and two sidewalks, is 20 000 lb. per. lin. ft., when all are completely loaded; and, if the structure were made of carbon steel, the dead load would be about 110 000 lb. per lin. ft., which would make the sections almost prohibitive. The designs, therefore, have been based on the use of eye-bar cables of alloy, heat-treated steel, having an elastic limit of not less than 120 000 lb. Such steel has already been manufactured for limited sizes, and as it is only here required for eye-bars, from which any number of full-sized specimens can be tested to destruction, there will be no doubt as to whether the strength and other qualifications are fulfilled. With such material, the total average dead load of the structure is 64 000 lb. per. lin. ft., so that this material makes a saving of 46 000 lb. per lin. ft., or practically 50% of the total load on the structure.

Such a material will naturally cost a considerable price, but it will be about four times as strong as the usual carbon steel; and even at much more than four times the cost, it would make a saving, on account of the large decrease in dead load.

About 40 000 tons of such bars will be required, practically all duplicates, so that there is little doubt that they will be furnished at a price which will make a great economy in the structure.

The speaker, therefore, fully agrees with Mr. Waddell that such high-value alloy steels are a necessity for coming bridge structures, and has the fullest confidence that our metallurgists will meet the demands as they arise.

CHARLES EVAN FOWLER, M. AM. SOC. C. E. (by letter).—For many years the writer has known of the higher grades of steel produced by European manufacturers, but, owing to the great demand on the mills of the United States, it has been a case of taking what is offered by the manufacturers or paying the extras asked for nickel steel or anything except the ordinary grades. The tariff conditions will now allow the importation of European high-grade steels for Pacific Coast fabrication, but, until shops of sufficient size are established there, little advantage can be taken of such foreign products. This possible source of supply may eventually result in American mills being willing to meet the demand for nickel or other high steel alloys at a reasonable price. Mr. Fowler.

In the writer's discussion* of Mr. Waddell's former paper, "Nickel Steel for Bridges", he called attention to the possible use of ferruginous nickel as a cheap means of obtaining the nickel ingredient for nickel steel, and it is gratifying to know that this has been declared possible

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 300.

Mr. Fowler.

by competent metallurgists and at a reasonable cost; so that now it seems to be only necessary to find philanthropic steel manufacturers who will make only a reasonable charge for manufacturing plates and shapes of the desired composition.

To conduct the extensive experiments which are desirable on nickel steel and other high steel alloys, it will certainly be necessary to enlist a really enthusiastic support from the mills before anything satisfactory can be accomplished. When it is possible to do this, and to raise the necessary funds, the work should be carried out under the direction of this Society, acting in conjunction with the American Society for Testing Materials.

The great amount of work done by Mr. Waddell, as shown by this and his former paper, will entitle him to a large share of the credit for the great spans which may be built in the future if, as a consequence, higher steels are made possible.

The diagrams of weights of metal in bridges, for both the usual and the higher grades of metal, show conclusively what one would naturally infer as to the great saving that may be made by the use of a steel of high elastic limit which can be obtained at a reasonable cost. It is to be regretted that all such investigations are not made on a common basis, both as to loadings and specifications, so that the work of various engineers along similar lines may be more readily compared, and the desired end be more quickly reached. To this end, the proper committee of the Society should formulate rules for all such investigations and calculations.

The engineer who has been in direct touch, not only with structures manufactured from his own designs, but also with those of hundreds of other engineers, as has been true in the writer's case at the shops of the Youngstown Bridge Company and at other large shops, and throughout a wide experience of more than a quarter of a century, will realize that the personal equation of the designer does result in such a wide variation in weights that, unless each one is tied down to the hard and fast rules of the same specifications, the resulting data cannot readily be compared, especially for long spans.

The real comparison can only come from the various designs that would naturally be made for any specific location and the investigation of the composition of the necessary high-grade metal required for that particular structure, as was the case for the St. Louis Eads Bridge. A structure costing from $30 000 000 to $60 000 000 would easily stand the charge of $100 000 or more for such experiments.

Long spans are seldom contemplated or built from motives of economy, but are the result of the necessities of commerce, of finding good foundations, or as necessary connecting links in lines of traffic and communication, regardless of whether the structure in itself will

Mr. Fowler.

be a paying investment. When, from some cause or other, long spans are found to be necessary, the engineer must determine the class of structure that will be possible:

First.—To carry the class and quantity of traffic to be imposed or cared for;

Second.—That will be possible, due to the foundations that can be obtained;

Third.—That will be possible, due to the material that can be obtained from which to fabricate it;

Fourth.—That will be possible, owing to the limitations that may be imposed by the methods and means of construction;

Fifth.—That will be a paying investment from a dividend-paying standpoint, if that be necessary;

Sixth.—That will at least be possible from related financial conditions.

These factors are correlated to such an extent that a very wide investigation would be necessary, in order to set the limits of span and expenditure.

Recent researches seem to indicate:

That simple spans can be constructed by using nickel steel to a greater or less degree, up to lengths of from two to three times those which have been built;

That suspension spans and cantilevers will reach about the same cost at some span length between 1 600 and 1 800 ft.;

That, by the use of nickel steel to a large extent, suspension spans can be built economically, where the traffic is sufficient, up to about 3 000 ft. span;

That, by the use of nickel steel to a large extent, cantilevers can be built economically, where the traffic is sufficient, up to about 2 500 ft. span.

However, inasmuch as the dead weight of such structures is very great in proportion to the live load, we can largely disregard impact, and use correspondingly high unit stresses, thus allowing at present cost the use of nickel steel for all the suspended structural parts of a suspension span, thereby making such a structure, with its high steel cables and comparatively low erection cost, the best to adopt for long spans, from every point of view, beyond a span length of about 1 700 ft. Such a structure will carry satisfactorily all classes of traffic, and satisfy most fully all the six requirements previously given.

The question of right of way and terminals may add so greatly to the cost of the project as to make its realization impossible.

In his discussion of Mr. Waddell's former paper, the writer called attention to the various cost factors that would come in to determine

Mr. Fowler.

the possible span length of any type, or indeed the type to be used in a particular location. The sub-factors of erection cost will also serve to decide these things. The empirical erection cost formula, devised by the writer some years ago, will serve to illustrate this point, there being six different factors which enter into the cost of only this one portion of the amount necessary to expend for the construction of a bridge superstructure:

$$C = a + \sqrt{l} + \frac{3}{4}h + \frac{200}{d} + 5p - \frac{1}{5}\sqrt{w - 500}$$

C = cost of erection, in cents per 100 lb.;
a = a constant for each type of structure;
= 15 cents for railway pin trusses;
= 25 cents for railway riveted trusses;
= 20 cents for railway girders;
= 20 cents for highway pin trusses;
= 30 cents for highway riveted trusses;
= 15 cents for highway girders;
l = span length, in feet;
h = height of falsework, in feet;
d = daytime temperature, average, in degrees Fahrenheit.

Taking the case of a riveted railway span 225 ft. long, 48 ft. height of falsework, average temperature, 40°, 2 coats of paint, and weighing 2 100 lb. per lin. ft., we find the probable erection cost to be 83 cents per 100 lb., or $16.60 per ton.

A railway pin span of 144 ft., 36 ft. height of falsework, average temperature, 50°, 2 coats of paint, and weighing 1 400 lb. per lin. ft., erection cost equals 62 cents per 100 lb., or $12.40 per ton.

These two examples show what an influence a change in any one of the factors will have on the unit erection cost. For cantilevers of the type of the Poughkeepsie Bridge, or the Knoxville Bridge, designed and built by the writer (Fig. 24), every other span being a fixed or anchor span, the bridge must have falsework, so that, for a bridge of this type one can take the coefficient of h at only $\frac{3}{8}$ instead of $\frac{3}{4}$. The formula was deduced to fit certain conditions, which to a large extent were due to the personal equations of the designer and the erector; and, with plans prepared by some designers, the cost of erection would exceed very greatly the values found from the formula, it being only too common on the part of many designers to forget that structures must be erected at a reasonable cost, and still others seem to forget the process of erection entirely.

Many erection costs, of course, will exceed greatly what they should, due to unforeseen causes. The White Pass, Alaska, Arch constructed

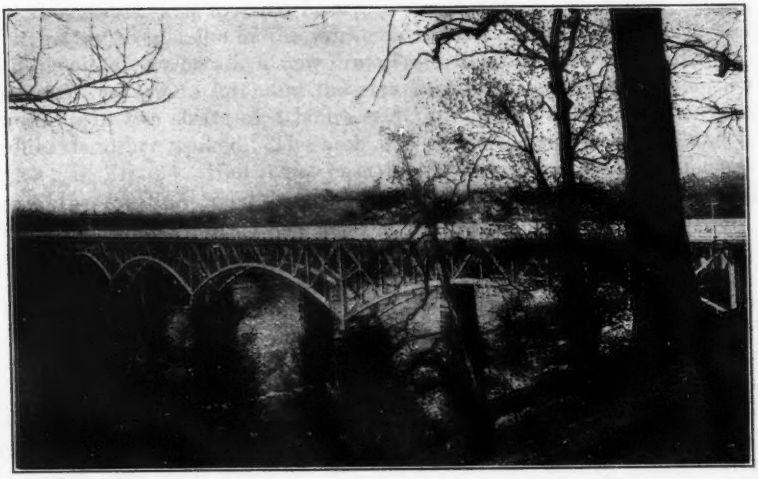


FIG. 24.—KNOXVILLE STEEL ARCHED CANTILEVER BRIDGE.

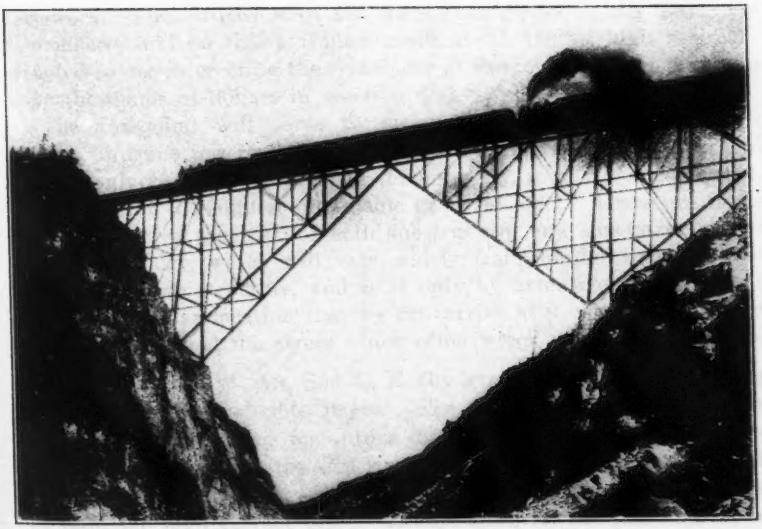
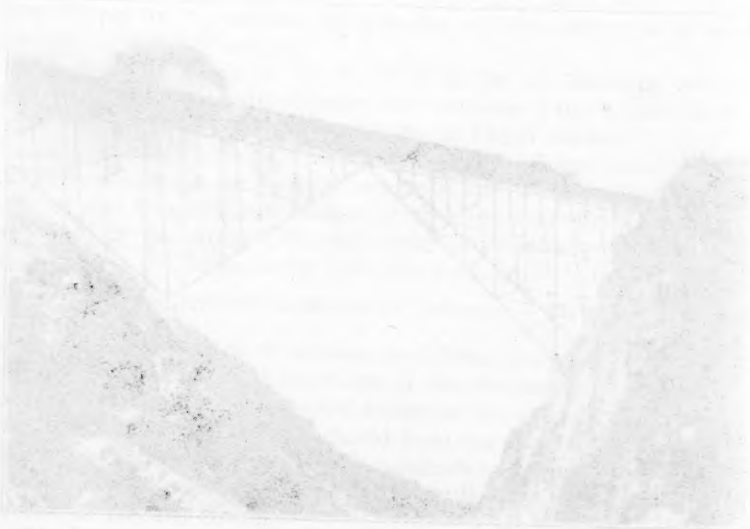


FIG. 25.—SWITCHBACK ARCH. WHITE PASS AND YUKON RAILWAY, ALASKA.



by the writer, was to have been completed before the winter season, but, owing to a strike of the workmen in September, and no telegraph line available to obtain a new crew from Seattle, 1 000 miles distant, some 6 weeks were lost, and the structure was completed by the men working in two shifts of 2 hours each, one crew warming up in camp while the other worked. The arch was completed in December, and is shown in Fig. 25. Had it not been for the delay and consequent extra cost, the erection could have been carried out on a very economical basis, as the falsework necessary for the anchor arms was very slight, and one top traveler carried out the entire work of erection. Mr. Fowler.

The extra cost due to delay in this or other cases, cannot, of course, be considered in making a design, but there are many features of erection methods which should be studied out during the original investigations. So far as the writer is aware, no scheme of erection was ever formulated for the construction of the Williamsburgh Bridge over the East River, on which he was Consulting Engineer for the erection of Manhattan tower and approaches. The Brooklyn tower erection had been planned and partly carried out by constructing trusses across from masonry pier to masonry pier, and these supported a temporary combination tower about 300 ft. high from which to erect the steel tower.

For the Manhattan end the tower was erected up to the floor level by stiff-leg derricks on a heavy bent at the end of the approach-span falsework. The latticed strut and floor-beam girder in the tower was also placed, and on this a timber tower about 150 ft. high was constructed to use in erecting the remainder of the tower steel, thus saving some thousands of dollars in erection cost.

The foregoing will serve to make perfectly plain the writer's reasons, in some few respects, for regarding diagrammed weights and costs as only very general proof of the necessity, in any particular case, for the use of the highest obtainable grade of steel. There are items in every part of the cost, of both substructure and superstructure of any great bridge, which will vary widely from similar items of cost in another large structure, and it is only by extensive investigations at each particular location that we can arrive at a close semblance of the truth and avoid the errors which often wreck meritorious projects.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is highly pleased with this valuable paper. The practical results given are exceedingly encouraging for future developments. The writer would respectfully suggest to the experimenters that they also try a small batch of aluminum steel. The main objection to its use, heretofore, has been the high market price. Lately, however, the price has come down to 20 cents per lb. in large lots. It is his firm belief that if the ordinary steel is first "thoroughly purified", and its elastic limit, Mr. Le Conte.

Mr. Le Conte. thereby, raised to 60 000 lb. per sq. in., and then a cake of aluminum, say 3%, be added to the molten metal, the result will be a highly satisfactory alloy, and at no great increase in cost per pound. The aluminum not only strengthens the steel enormously, but also toughens it and wipes out all brittleness, and thus renders all general shop work much easier and more satisfactory and reliable in every way.

In case some of the material should be condemned as not coming up to the requirements of the specifications, the condemned material could easily be turned over to the wire works which would be only too glad to get it, as this aluminum steel makes the finest kind of high-grade wire, suitable in every respect for suspension bridges, as it is exceedingly strong, tough, free from brittleness, and non-corrosive to a high degree. Ordinary sections of this alloy will have an elastic limit of 100 000 lb. per sq. in., and the wire possibly 200 000 lb. per sq. in.; and at a cost probably not exceeding that of ordinary carbon steel by more than 2.5 cents per lb.

Mr. Petinot. N. PETINOT,* ESQ.—The speaker agrees with Mr. Waddell that the first step is to experiment on the purification of steel so that the metal may be brought to its maximum efficiency. He cannot see why it should not be possible to secure regularly physical properties within a limit of 5% in a steel carrying predetermined quantities of carbon and nickel, for instance.

The speaker has been experimenting extensively, both in France and in the United States, with the manufacture of alloy steels, and particularly with nickel steels of the same supposed analysis, and has found that very often two melts of open-hearth nickel steel of the same analysis, when physically tested, have shown a variation of 15% or more. Carrying the investigation further he found, by microscopic examination, a large difference between the two steels. The poorer very often showed very large colonies of manganese sulphides, iron silicates, and slag. These are weakening elements in steel of any grade, even if it is an alloy of plain carbon metal.

When the manufacturers of steel find the way to remove such elements, it will be possible to secure the maximum physical qualities from each component of every grade of steel, and there should be no great difference between any two steels of the same chemical composition.

Increasing the carbon content in steel of any grade is a cheap way to increase the alternate strength, and, according to the speaker's experience, a satisfactory way, provided the steel has been thoroughly cleaned and the segregation reduced to the minimum.

In reference to the possibility of producing a nickel steel by the use of a ferro-nickel containing 10% of nickel, this problem presents

* Metallurgist, The Titanium Alloy Manufacturing Company, Niagara Falls, N. Y.

Mr. Petinot.

no difficulties at all, as such steels have already been produced in 5-ton melts. A few years ago a pig iron was produced by smelting pyrrhotite—containing nickel—in an electric furnace. This pig iron contained from 4 to 5% of nickel and was low in sulphur and phosphorus. By varying the proportions of such pig iron in the mixture used in making nickel steel by the open-hearth process, it will be possible to obtain any nickel content actually desired.

It will be noticed that, in the making of nickel steel, the nickel is usually added with the cold charge, and does not, therefore, oxidize during the periods of melting and finishing the steel, and its use as a component of pig iron would be the same under similar conditions.

The speaker believes that, by experimenting thoroughly with ferro-nickel in the manufacture of nickel steel, it will be possible to get a product which will be identical with that obtained by the use of metallic nickel, and at a much lower cost.

Mr. Waddell has asked what effect the use of titanium might be expected to have on the steel. Titanium has a great affinity for nitrogen and oxygen, so much so that, prior to this time, it has been very difficult to obtain a steel with a content of more than 0.25 of that metal, and such steel showed no particular advantages over one which had been treated with 0.10 of titanium, and after such treatment carried only 0.02 to 0.03 of the latter.

Titanium is on the market commercially in the form of ferro carbon-titanium, containing from 15 to 20% of titanium and from 6 to 8% of carbon. It has been found that a ferro-titanium, with a higher titanium content, has too high a melting point for general use.

Titanium, because of its affinity for nitrogen and oxygen, the latter either in the form of a gas or an oxide, is to be considered as a powerful deoxidizer and scavenger. It is the only deoxidizer known at present which can be used without danger of leaving any of the products of its oxidation in the bath of steel.

In the treatment of steel with titanium, the ferro carbon-titanium is always used as the last addition to the steel after it has been tapped into the ladle, that is, after manganese, silicon, or other alloys, if any, have been added. Titanium will react first on nitrogen (always present in a greater or less quantity), forming titanium nitrides (Ti_2N_2), which rise to the top of the ladle. It will then act on the oxygen in solution in the steel, and on oxides of iron, manganese, etc.

Particular attention is called to the presence of manganese oxide in steel. Ferro-manganese is added to the steel, not only as a deoxidizer, but also to furnish sufficient manganese to combine with the sulphur, giving manganese sulphides, which are less brittle than iron sulphides. It has always been found that the rolling properties of steel are enhanced by a small manganese content, and it is a well-known fact

Mr. Petinot.

that it is impossible to roll a steel ingot which does not contain manganese.

Now consider, for instance, what happens when manganese is added to molten steel. The iron oxides, for instance, are robbed of their oxygen, and manganese oxides are formed, as shown by the following chemical equation:

$$FeO + Mn = Fe + MnO,$$

but, as manganese oxide will partly remain in the steel, if a piece of this steel is analyzed, the chemical laboratory will report so much manganese, but there will be nothing to show in what form this manganese occurs, whether as an oxide, sulphide, or some other compound.

It is difficult to determine whether a remainder of manganese oxide in the steel is more or less harmful than a content of iron oxide, which it may have replaced. In the manufacture of soft steel, with a carbon content of from 0.08 to 0.10 and manganese content of 0.40, it has been proved that, by eliminating the manganese oxide completely, no blisters will be found when such steel is rolled into sheets and galvanized.

When titanium is added to steel it will react as shown by the following equations:

On nitrogen:

$$Ti_2 + N_2 = Ti_2N_2 \quad \ldots\ldots (1)$$

On oxygen:

$$Ti + O_2 = TiO_2 \quad \ldots\ldots (2)$$

On iron oxide:

$$2FeO + Ti = 2Fe + TiO_2 \quad \ldots\ldots (3)$$

On manganese oxide:

$$2MnO + Ti = 2Mn + TiO_2 \quad \ldots\ldots (4)$$

The titanic oxide (TiO_2) formed by any of the last three reactions will flux all particles of slag invariably suspended in the steel and carry such slag to the top of the ladle, which will enable steelmakers to teem into the ingots a product, practically free from nitrogen oxides and slag, which are the weakening elements in steel of every grade.

Another function of titanium is to reduce segregation to its minimum. The following explains briefly how segregation is produced: Steel must be considered as a mixture of pure iron, iron carbide, manganese sulphides, iron silicides, iron phosphides, etc., each of the components having different specific gravities, and varying melting points. Now, consider what will happen when an ingot mould is filled with molten steel: The wall of the mould will act as a chill, and the component having the highest melting point will be the first to solidify. If drillings are taken, starting from the outside of the ingot toward its center, it will be found that the carbon and other elements of lower specific gravity and higher melting points will be

higher inside the part cross-hatched in Fig. 26 and also at the top of the ingot. Mr. Petinot.

Now, consider why the various elements composing the steel will segregate, and why the maximum segregation will be found at the top of the ingot. This segregation is produced by the presence of oxides in the steel, no matter whether they are of manganese, of iron, or other elements. Take, for instance, a molecule of oxide, *R* (Fig. 26). The carbon of the steel will react on this molecule, giving carbon monoxide, according to the following equations:

(In the case of iron)

$$FeO + C = Fe + CO \quad \ldots\ldots(5)$$

(In the case of manganese)

$$MnO + C = Mn + CO \quad \ldots\ldots(6)$$

The tendency of this carbon monoxide gas is to release itself from the steel by rising to the top of the mould. The bulk of this gas has been produced in that part of the ingot which is last to solidify, as previously mentioned. This part of the ingot is composed primarily of iron carbides, iron phosphides, manganese sulphides, etc., so that all these elements will be found in excess near the top.

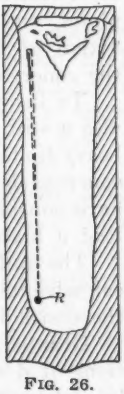


FIG. 26.

It very often happens that a steel which shows in a ladle test 0.40 carbon will show, when rolled into bars, a variation of carbon between 0.35 and 0.45. If titanium is added, it will prevent, to a great extent, the formation of CO in the moulds, because, as already shown by Equations 3 and 4, the titanium will react on these oxides in the ladle, so that, when the steel is poured into the moulds, the only segregation that will occur is that caused by the chilling action of the moulds.

The speaker's experience has been that a clean product, with very little segregation, can be obtained by the use of titanium, at an additional cost of from 25 to 50 cents per ton of steel treated, and, if this is so, the uniform composition of such a steel will easily warrant small additional expenditure.

LEON S. MOISSEIFF, M. AM. SOC. C. E.—This paper is most interesting and also most timely. It attempts to find a way to meet the ever increasing demands made on the ingenuity of the engineer and the resources of the capitalist by heavier loads and greater spans. There is no doubt that the author points the right way into the future: there is a need of a material of a higher strength than that of the steel generally used in bridge building to-day. Whether it be nickel steel, as recently used in some of our long-span bridges, or another kind of an alloy steel, still better meeting the requirements of bridge building, there is a technical need of a higher strength steel. Not only would Mr. Moisseiff.

Mr. Moisseiff.

heavy railroad bridges with their increasing freight loads be benefited by the higher steel, but also highway bridges of the longer spans.

The advent of heavy automobile trucks has made it imperative to provide heavy floors for bridge roadways. The old rule, of an inch depth of plank for every foot of span for the usual timber floor, died quite some time ago. Nowadays, concentrations of 10 000 to 15 000 lb. have to be taken care of on city highways. Even the old reliable buckle-plate, built in a suspended position, appears to have lost its good behavior. Recently, it has been found on some highway bridges that the buckle-plates are not sufficiently rigid, and that the pavement fails in consequence. This can readily be explained. The suspended buckle-plates are stiffened by the filling of concrete placed on them. The stiffness thus provided was sufficient for the old-time concentrations from wheel loads, and the pavement was supported rigidly enough to give good results. The increased concentrations of modern traffic, however, overcome the stiffening resistance of the concrete and the buckle-plates, and the curve of the plate is distorted under the advancing action of the load. This results in the crushing and crumbling of the concrete in the trough, and the ultimate failure of the pavement.

To insure the good behavior of a pavement, not only a strong but also a stiff support is necessary. Such support will be furnished by a heavy timber floor with floor stringers at close intervals, or by reinforced concrete slabs. In either case a floor system will be required which will have considerable weight. To take care of this additional weight and load a high-strength steel is wanted.

The author has gone to a great deal of trouble to show the economical advantage of higher steel, and there can remain no doubt as to the desirability of its use for bridges.

Granted, then, the desirability of the higher strength steel, questions of design may well be considered. One of these questions may be: what will be the effect of the higher steel on the stiffness and stresses of bridges built of that material?

The higher elastic limit of the new steel will be utilized, of course, for allowing higher unit stresses, which is the purpose of its use. The coefficient of elasticity being practically constant for kindred steel, the higher unit stresses will result in greater elongations of the individual members and greater deflections of the entire structure. The increased deformation of the trusses means a greater deviation from the original truss-form, and will result in much increased secondary stresses.

Assuming, as the author has apparently in the paper, that the allowable unit stresses will retain the same proportion to their corresponding elastic limits as they have in the present practice in the case of carbon steel, the allowable stresses for the hoped for high-alloy steel of 100 000 lb. elastic limit will be nearly three times those of carbon steel with its elastic limit of 35 000 lb. Evidently, the moving load deflections will

become considerable and the secondary stresses may attain the rank of primary stresses. How, then, to take care of the secondary stresses in the erection of the bridge, and provide for the deformation of the trusses under moving load, will raise new and serious problems in bridge design. Mr. Moisseiff.

Another question is how to connect efficiently the high steel members, apart from using pin connections. With common soft steel rivets the splices would be of excessive length, the efficiency of which is subject to much doubt. Consequently, high-steel rivets must be used. Nickel steel members have been designed by the speaker for corresponding nickel steel rivets, but these rivets have not always proved very satisfactory. They are not easily well driven and are difficult to remove. This refers especially to field rivets which have to be driven with a gun or a machine of limited pressure. Now, if nickel steel rivets having an elastic limit of 50 000 lb. present some difficulties, what will rivets of, say, 80 000 lb. elastic limit offer?

F. W. Skinner, M. Am. Soc. C. E.—The speaker, having recently had an opportunity of seeing some phenomenally large carbon steel rivets, made for the new Quebec Bridge, would like to confirm to some extent the views of the author and of Mr. Hodge. Some of these rivets—those for the cantilever arms—were 7 in. or more in length, with a diameter of 1¼ in. Mr. Skinner.

Some experimental work was done in which these rivets were specially designed with a slight taper at the points. They were quenched at the points in cold water. When driven, they were held in place by a pneumatic bucker, and the driving was done with a pneumatic hammer. The pneumatic bucker was a special tool, having a hollow cylinder in which a hammer was arranged. The hammer and cylinder were controlled independently, which enabled the bucker to be held in position, and then when the ordinary hammer was operated on the point of the rivet, the secondary hammer in the bucker was operated, and the rivet was thus driven simultaneously from both ends. The result was that they were exceedingly well driven. A row of rivets was driven partly by yoke machines and partly by the hammer and bucker process, as in the field. When they were cut in two, it was almost impossible to distinguish between them, the field-driven rivets being, for every practical purpose, as good as those driven by machine in the shop. Therefore, it seems quite reasonable to expect that field-driven rivets can be made substantially as good as shop rivets, and, if nickel steel rivets can be driven satisfactorily, it may go a long way toward producing equally good results in the field.

Regarding the possibilities of high alloy steel in bridges, there can be no doubt that the need of it is great and the prospect is alluring. The speaker supposes that it is generally agreed that there is difficulty

Mr. Skinner.

in securing uniformity in its production. A great many years were required to produce carbon steel which was trustworthy and uniform, and it will no doubt take as long to get alloy steel in uniform condition. The speaker would be very glad to hear from the author about the possibilities of developing a super-strong alloy steel by subjecting finished members to heavy test loads at the shops. It has been pretty well demonstrated that both the elastic limit and ultimate strength of carbon steel, for both tension and compression, can be thus greatly raised. The speaker does not know that this has gone farther than a demonstration, but it was considered more or less seriously for the Quebec Bridge, and perhaps for the St. Louis Municipal Bridge. Such treatment should achieve the double result of increasing the elastic limit and ultimate strength, and therefore the working capacity, and should serve as a valuable test or proof load, disclosing all serious defects.

If an increase of from 20 to 40%—possibly even more—can be practically attained at moderate expense for carbon steel, what would be the result if applied to nickel or other alloy steel? It may be inferred that important results can be obtained. The speaker thinks that it opens up a wide field of conjecture.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE ALFRED NELSON, M. Am. Soc. C. E.*

DIED JUNE 3D, 1913.

George Alfred Nelson, the son of George and Abigail Marion Bigelow Nelson, was born in Lincoln, Mass., on September 20th, 1852. He attended the Lexington District schools and the Lincoln High School and was graduated from the latter in 1872. In 1873, he entered the Massachusetts Institute of Technology from which he was graduated in Civil Engineering in 1877. During a summer vacation, from June to September, 1875, he served as Rodman on the survey and construction of the Boston, Concord and Montreal Railroad, and as an Assistant in the office of the City Engineer, at Concord, N. H.

After his graduation from the Massachusetts Institute of Technology, Mr. Nelson spent two years at his home in Lincoln, and was engaged in surveying and various engineering works in that vicinity. In 1879, he went to Lawrence, Mass., where he was employed as sketch maker in the Designing Department of the Pacific Mills. He remained in this position until August, 1883, when he resigned to become Assistant Engineer in the office of the City Engineer at Lowell, Mass., which position he held until his death.

As Assistant Engineer, Mr. Nelson had charge of the design and construction of several bridges in Lowell, one of which was the Taylor Stone Arch Bridge across the Concord River, the location of which involved difficult foundations, and its successful completion showed the thought and skill devoted to its design. He also had charge of a complete survey of the city for assessors' maps; of all water-works improvements; the extension of the sewerage system into new territory; and the design and cost estimates for the abolition of grade crossings within the city limits.

Mr. Nelson was a man of strong character and his personality and keen mind impressed all with whom he came in contact. He had never been very strong physically, and his constant and close attention to the details of his work affected his health so that he was forced to seek rest at frequent intervals. His death occurred on June 3d, 1913, after an illness of a few months. Mr. Nelson had never married. He is survived by a sister and two brothers.

He had always shown much cleverness with his pencil and crayon, and was an expert photographer. He had shown his photographs at

* Memoir prepared by the Secretary from material on file at the Society House.

various exhibitions in the United States, receiving many medals for his artistic work. With a few others, he was selected to represent the United States at an international exhibition at Berlin, Germany, where his photographs won for him a silver medal.

Mr. Nelson was an active member of the Eliot Congregational Church Society, at Lowell, and for many years was President of the John Eliot Literary Society, contributing greatly to the success of its work by his active energy and personality. He was a member of the Alumni Association of the Massachusetts Institute of Technology, the Technology Club of the Merrimac Valley, of which he was an active president for several years, and the Association of the Class of 1877 of the Massachusetts Institute of Technology. He was also a member of the Boston Society of Civil Engineers, having designed the pin which was adopted by that Society. He was devoted to outdoor sports and took an active part in the snowshoe trips of the Appalachian Mountain Club, of which he was a member. He was also a member of the Vesper Country Club, at Lowell, and an expert golfer.

George Alfred Nelson was elected a Member of the American Society of Civil Engineers on April 4th, 1911.

BENJAMIN FRANKLIN MORSE, M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1914.

Benjamin Franklin Morse was born on June 7th, 1829, in South Kirtland, Geauga County, now Lake County, Ohio, and in 1836 became a resident of Painesville, in the same county. During his residence at Painesville, he attended the common schools and afterward the Painesville Academy; later, he studied mathematics and civil engineering with General E. A. Paine, a graduate of West Point, who had retired from the United States Army.

Mr. Morse was identified with much of the pioneer railroad building in Northern Ohio, in the capacity of Assistant Engineer, in which connection he represented the Lake Shore Railroad, aiding in the construction of the line between Cleveland and Erie, Pa., in 1851 and 1852.

In 1853, he was Assistant Engineer, under Major Potter, at the harbors of Fairport, Ashtabula, and Conneaut, Ohio. During the season of 1854 he was First Assistant, under Captain Howard Stansbury, U. S. Engineer, in the examination of the harbors on Lake Erie west of Cleveland, namely Lorain, Vermilion, Huron, Sandusky, and Monroe, Mich.

* Memoir prepared by J. F. Morse, Esq.

As First Assistant Engineer, Mr. Morse had charge of a line from Tiffin, Ohio, to Fort Wayne, Ind., now a part of the Nickel Plate System. During 1855 he was Assistant Engineer in charge of a survey for The Cleveland and Mahoning Railroad, from Youngstown, Ohio, to New Castle, Pa., and from 1857 until 1862, he was First Assistant, under Mr. Charles Collins, in the Engineering Department of the Lake Shore Railroad, between Cleveland and Erie, Pa.

In 1862 the four railroad companies, the Cleveland and Columbus, the Cleveland, Painesville, and Ashtabula, the Cleveland and Toledo, and the Cleveland and Pittsburg, proposed through their Presidents to build the present Union Station in Cleveland. The Presidents constituted the Building Committee, with Mr. Amasa Stone, of the Lake Shore, as Chairman, and he appointed Mr. Morse as his Engineer. The latter drew the plans for the station, and they were approved by Mr. Stone. They included what was probably at that time one of the largest train-sheds in the United States. Mr. Morse superintended the building of the Union Station, completing the work in 1865.

In 1868, as Chief Engineer, he surveyed the line from Chardon to Youngstown, Ohio, which is now a branch of the Baltimore and Ohio Railroad, extending from Fairport to Youngstown. As Chief Engineer, he also made a preliminary survey for a railroad from Cleveland to Sharon, Pa.

Mr. Morse afterward became interested in the erection of many of the public buildings in Cleveland and superintended their construction. He superintended the construction of the City Work House and drew plans and superintended the rebuilding of the Newburg State Hospital, which was destroyed by fire in 1872.

In April, 1875, Mr. Morse was appointed City Engineer of Cleveland, in which capacity he served for 9 years. He remodeled the plans and completed the Superior Street Viaduct, which was opened in 1878. While acting as City Engineer, he estimated and reported on several high-level bridges, and one of the plans he advocated, the Central Viaduct, was adopted and developed.

He also first suggested and planned for the intercepting sewer for the City of Cleveland. He was appointed by the Building Committee to superintend the construction of the Society for Savings Building, but before active work was begun, he engaged with the Lake Shore and Michigan Southern Railroad Company to look after the building and rebuilding of its stations at Toledo and Chicago.

In 1890, under the new Building Code, Mr. Morse was appointed Building Inspector, and served in that capacity for 3½ years. After retiring from this office he spent some time in travel, and laid aside active business except for occasional consultation work in engineering lines.

On September 19th, 1913, Mr. Morse was seriously injured in an automobile accident, from the effects of which he died on February 17th, 1914, at the Battle Creek Sanitarium where he had gone for treatment.

In 1855, Mr. Morse was united in marriage to Matilda Craft, of Tiffin, Ohio. He is survived by three sons and one daughter.

He was a Royal Arch Mason, and for many years belonged to the old Board of Trade, and also to the Chamber of Commerce, of Cleveland.

Mr. Morse was one of the oldest members of the American Society of Civil Engineers, having been elected a Member on July 12th, 1877. He was a Charter Member of the Civil Engineers Club of Cleveland, now the Cleveland Engineering Society.

PHILIP CHAPIN DAVIS, Assoc. M. Am. Soc. C. E.*

DIED MARCH 26TH, 1914.

Philip Chapin Davis was born on September 14th, 1881, at Kalamazoo, Mich., in which city he obtained his early education. In 1906, he was graduated from the Engineering Department of the University of Michigan, with the degree of Bachelor of Science in Mechanical Engineering. In September of that year, he entered the employ of the Thompson-Starrett Company, General Builders, and was with that company on several notable engineering problems, both in Chicago and New York, until August, 1911, when he became associated with the Jobson-Gifford Company of New York, as Superintendent of Construction in connection with the electrification of the New York, New Haven and Hartford Railroad, between New York City and New Rochelle, and also between Stamford and New Haven.

On October 29th, 1908, Mr. Davis was married to Miss Bertha Shean, of Kalamazoo, Mich., who, with two sons, survives him. Mr. Davis died at the Neurological Institute, New York City, on March 26th, 1914, following a year's illness from recurring attacks of pernicious anæmia.

Mr. Davis was a member of Anchor Lodge, No. 87, F. and A. M., and a member of the Sigma Chi Fraternity. He was a young man of exceptional ability, and his friends and associates feel deeply their loss on account of his untimely death.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on June 4th, 1913.

* Memoir prepared by Erle K. Knight, Assoc. M. Am. Soc. C. E.

ROGER TIFFT HOLLOWAY, Assoc. M. Am. Soc. C. E.*

DIED MARCH 12TH, 1914.

Roger Tifft Holloway, the second son of Henry F. and Metta J. Holloway, was born on August 29th, 1885, in Columbus, Ohio. In 1887, the family moved to Montclair, N. J., and he was educated at the public schools of that place. He was graduated from the High School in 1904, and was President of his class during his Junior and Senior years. In the fall of that year, he entered Cornell University, from which he was graduated in 1908, with the degree of Civil Engineer. While at Cornell, Mr. Holloway was elected a member of the Alpha Delta Phi and of the Sphinx Head, a Senior society, and he also belonged to the Cornell Glee Club.

In August, 1908, Mr. Holloway was engaged by the Turner Construction Company, of New York City, as Assistant to the Superintendent on reinforced concrete work, remaining with the Company until January, 1909. From March to October, 1909, he was employed as Structural Detailer and Draftsman with the Hay Foundry and Iron Works, at Newark, N. J. In February, 1910, with the writer, he engaged in private practice, as a Consulting Structural Engineer, under the firm name of Mead and Holloway, continuing as a member of the firm for two years. During this time he prepared plans, estimates, etc., for numerous alterations and contracts, among which were plans and specifications for the steelwork for the Irvington School, the Nurses' Home in connection with the Presbyterian Hospital, in Chicago, Ill., the Dock Street Pier, Philadelphia, Pa., etc. He also acted in an advisory capacity on this work.

In June, 1912, Mr. Holloway severed his connection with the firm of Mead and Holloway to engage in the general practice of civil engineering. He was also the New York representative of the London firm of Bagley, Mills and Company, and until his sudden death which occurred at his home in Montclair on March 12th, 1914, from septic poisoning, following an attack of tonsilitis, he was engaged in preparing plans, specifications, estimates, etc., for a number of constructions and alterations in and around New York City.

Mr. Holloway was always courteous, and, in his short professional career, had established a reputation for painstaking care and accuracy which won him many friends. The aptitude shown by him in his profession was probably inherited, his father being a member of the American Society of Mechanical Engineers, and a great-uncle, Mr. J. F. Holloway, having served a term as President of that Society.

* Memoir prepared by Charles A. Mead, M. Am. Soc. C. E., supplemented by information on file at the Society House.

He was a member of the University Glee Club of New York, the Harlequins, of which he was President, the Montclair Athletic Club, the Alpha Delta Phi Club of New York, and Montclair Lodge No. 144, F. and A. M.

Mr. Holloway was prominent in the social activities of Montclair where his loss will be keenly felt. His engagement had recently been announced, and the wedding had been planned for the coming Autumn. Besides his parents, he is survived by two brothers and two sisters.

Mr. Holloway was elected a Junior of the American Society of Civil Engineers on May 31st, 1910, and an Associate Member on May 7th, 1913.

JOHANNES CORNELIS VLIEGENTHART, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 29TH, 1913.

Johannes Cornelis Vliegenthart was born in Delft, Holland, on July 15th, 1876. In 1893, he entered the Polytechnical School of Delft as a student of engineering, and was graduated therefrom in July, 1899, with the degree of Civil Engineer.

In September, 1899, he entered the Government service of The Netherlands as Assistant Engineer in the Royal Corps of Waterstaat, in charge of river improvement works of the waterway from the Port of Rotterdam to the sea, at Hook of Holland and Hansweert. This work included dredging, constructing groins for regulating the channel, etc.

In November, 1901, Mr. Vliegenthart was appointed Chief Engineer of the Haiho River Conservancy Commission, with headquarters at Tientsin, China. This Commission was appointed by the Peace Protocol after the Boxer Insurrection of 1900, for the purpose of dredging and regulating the Haiho River, with a view to making it navigable from Tientsin to the sea for steamers drawing from 10 to 14 ft. of water. Under his direction three channels were constructed of a total length of 4 miles, thereby shortening the river by 15 miles and eliminating ten sharp bends. In addition to this work, Mr. Vliegenthart had charge of a survey of part of the Province of Chili, which included the cities of Peking, Tientsin, and Paoting. This survey was made chiefly for the purpose of studying the courses of the various tributaries of the Haiho River, looking to the improvement of such tributaries. He had promised to read a paper before the Koninklijk Instituut van Ingenieurs (Royal Institution of Netherland Engineers) on the great engineering works in North China in which he had had such a prominent part, but the fulfillment of this promise was prevented by his death.

* Memoir prepared by the Secretary from information on file at the Society House.

With the exception of a vacation in 1906, he remained in China until early in 1913, when he resigned his position with the Haiho Conservancy Commission and returned to his home in Holland. In September, 1913, on the election of the well-known engineer, Mr. R. R. L. de Muralt as a Member of Parliament, Mr. Vliegenthart was appointed to succeed him as Engineer of the "Waterschap Schouwen" (Hydraulic Company, Isle of Schouwen, Province of Zeeland). He remained in this position until his death which occurred on October 29th, 1913, at Zierikzee, Holland.

He belonged to that group of Netherlanders who have raised the reputation of Dutch engineers in foreign countries to a high standard, and his name will be held in honored remembrance as one of the pioneers who co-operated in the development of China.

Mr. Vliegenthart was elected an Associate Member of the American Society of Civil Engineers on June 5th, 1907.

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EXTERNAL CORROSION OF CAST-IRON PIPE.

By Marshall R. Pugh, M. Am. Soc. C. E.

To be Presented October 7th, 1914.

Synopsis.

Cast-iron pipe is in such general use that a knowledge of its strong points, its limitations, and the precautions necessary to insure its durability under certain conditions, is of the highest importance to the engineer.

The purpose of this paper is to warn the engineer of conditions where special protection is needed against external corrosion, and to suggest preventive measures adapted to different cases, in the hope of bringing out more information in the discussion.

Tuberculation and electrolysis from stray currents have been studied elsewhere at length and will not be discussed herein.

Examples of the great durability of cast-iron pipe are first given, after which other instances are cited in which it has deteriorated rapidly. Mention of the composition of cast iron and the theories of its corrosion is followed by a study of conditions contributing to, and inhibiting, corrosion.

In the light of this information, reasons are sought for the de-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

terioration observed in typical cases; and then precautionary and preventive measures are considered.

Introductory.

Records of the facts relating to the corrosion of cast-iron pipe are so scattered (frequently consisting of incidental references in articles on unrelated branches of engineering), and so much of the literature is found only in publications familiar chiefly to the chemist and physicist, that, abandoning any effort at originality, it seemed worth while to gather these scattered facts and what little is known, in the hope that additional information might be brought out and light thrown on many perplexing problems; and that the whole might be placed before the engineer in accessible form.

"There is no limit to the life of cast-iron pipe", says a recent advertisement in a technical journal, and this undoubtedly voices the opinion of many well-informed persons. Opposed to that we find the following: "The relatively high rate of self-corrosion [as distinguished from electrolytic corrosion] of cast iron as compared to the other kinds of iron tested is contrary to the generally accepted idea that cast iron is more resistant to self-corrosion than wrought iron."*

This paper will be confined entirely to a study of the external corrosion of cast-iron pipe, though, obviously, many facts must be applicable as well to internal corrosion. As stated, electrolysis from stray currents has already been dealt with so fully that it will not be taken up. Tuberculation has been studied exhaustively,† and internal corrosion and hot water troubles have been investigated very ably.‡

The ancients were unacquainted with pipe of this material, as the art of casting iron was unknown until the 14th or 15th Century. The Romans, in the time of the Cæsars, used pipes of stone, bronze, and lead. Fig. 1 shows a Roman, bored stone pipe, and Fig. 2 a lead pipe, with a section the actual size of the pipe. These pipes were always oval in shape, and were made of a sheet of lead bent

* "Electrolytic Corrosion of Iron in Soils," by Burton McCollum and K. H. Logan, Technologic Papers, U. S. Bureau of Standards, No. 25.

† "Tuberculation and the Flow of Water in Pipes", by Nicholas S. Hill, Jr., *Proceedings*, Am. Water Works Assoc., 1907, p. 303.

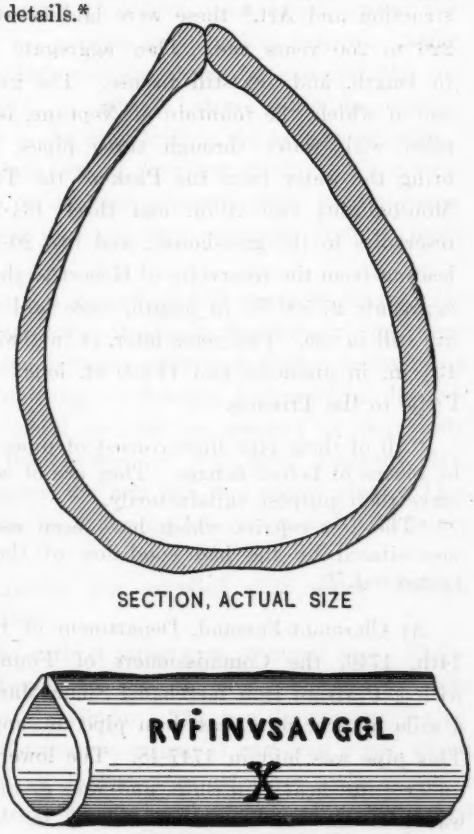
‡ "Hot Water Troubles", by George C. Whipple, M. Am. Soc. C. E., *Proceedings*, Am. Water Works Assoc., 1911, p. 231.

up and the edges soldered together. Sextus Julius Frontinus, Water Commissioner of Rome (*Curator Aquarum*) from 97 A. D. to 106 A. D., wrote two books describing the works under his charge, which are full of the most interesting details.*

In more modern times, London, in 1236, was provided with water from Tyburn, which was brought to the city through a lead pipe. The water rights were purchased by the magistrates from Gilbert Sandford.

ANCIENT ROMAN
BORED STONE PIPE

FIG. 1.



ANCIENT ROMAN PIPE
MADE FROM SHEET LEAD

FIG. 2.

EXAMPLES OF DURABILITY OF CAST-IRON PIPE.

We cannot go back so far in our study of the life of cast-iron pipe. The earliest records which appear to be authentic relate to several old pipes of various diameters laid, by order of Louis XIV, near

* "The Two Books on the Water Supply of the City of Rome, by Sextus Julius Frontinus", translated by Clemens Herschel, M. Am. Soc. C. E., Boston, Dana, Estes & Co., 1899.

terioration observed in typical cases; and then precautionary and preventive measures are considered.

Introductory.

Records of the facts relating to the corrosion of cast-iron pipe are so scattered (frequently consisting of incidental references in articles on unrelated branches of engineering), and so much of the literature is found only in publications familiar chiefly to the chemist and physicist, that, abandoning any effort at originality, it seemed worth while to gather these scattered facts and what little is known, in the hope that additional information might be brought out and light thrown on many perplexing problems; and that the whole might be placed before the engineer in accessible form.

"There is no limit to the life of cast-iron pipe", says a recent advertisement in a technical journal, and this undoubtedly voices the opinion of many well-informed persons. Opposed to that we find the following: "The relatively high rate of self-corrosion [as distinguished from electrolytic corrosion] of cast iron as compared to the other kinds of iron tested is contrary to the generally accepted idea that cast iron is more resistant to self-corrosion than wrought iron."*

This paper will be confined entirely to a study of the external corrosion of cast-iron pipe, though, obviously, many facts must be applicable as well to internal corrosion. As stated, electrolysis from stray currents has already been dealt with so fully that it will not be taken up. Tuberculation has been studied exhaustively,† and internal corrosion and hot water troubles have been investigated very ably.‡

The ancients were unacquainted with pipe of this material, as the art of casting iron was unknown until the 14th or 15th Century. The Romans, in the time of the Cæsars, used pipes of stone, bronze, and lead. Fig. 1 shows a Roman, bored stone pipe, and Fig. 2 a lead pipe, with a section the actual size of the pipe. These pipes were always oval in shape, and were made of a sheet of lead bent

* "Electrolytic Corrosion of Iron in Soils," by Burton McCollum and K. H. Logan, Technologic Papers, U. S. Bureau of Standards, No. 25.

† "Tuberculation and the Flow of Water in Pipes", by Nicholas S. Hill, Jr., *Proceedings*, Am. Water Works Assoc., 1907, p. 303.

‡ "Hot Water Troubles", by George C. Whipple, M. Am. Soc. C. E., *Proceedings*, Am. Water Works Assoc., 1911, p. 231.

up and the edges soldered together. Sextus Julius Frontinus, Water Commissioner of Rome (*Curator Aquarum*) from 97 A. D. to 106 A. D., wrote two books describing the works under his charge, which are full of the most interesting details.*

In more modern times, London, in 1236, was provided with water from Tyburn, which was brought to the city through a lead pipe. The water rights were purchased by the magistrates from Gilbert Sandford.

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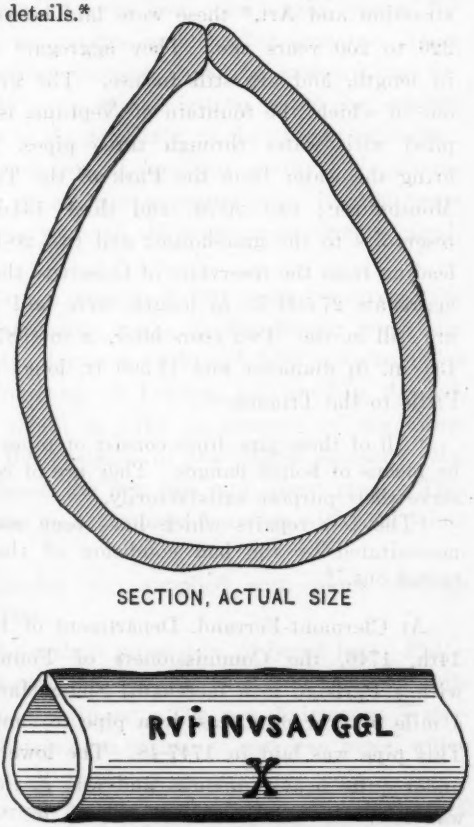

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Paris, from the reservoirs of Picardie to those of Montbauron, together with the spring water conduit, the whole supplying the town and parks of Versailles. According to the Ministry of Public Instruction and Art,* these were laid between 1664 and 1688, or from 226 to 250 years ago. They aggregate about 8 000 m. (26 000 ft.) in length, and are still in use. The great fountains at Versailles, one of which, the fountain of Neptune, is shown in Fig. 10, are supplied with water through three pipes, 20 in. in diameter, which bring the water from the Park of the Trappists to the reservoirs of Montbauron; two 20-in. and three 13½-in. pipes leading from the reservoirs to the gate-house; and two 20-in. pipes and a 13½-in. pipe leading from the reservoirs of Gobert to the reservoirs of l'Aile. These aggregate 27 500 ft. in length, were laid in 1685, 229 years ago, and are still in use. Two years later, or in 1687, the conduit of Chevreloup, 13½ in. in diameter and 11 500 ft. long, was laid from the reservoir l'Aile to the Trianon.

"All of these pipe lines consist of pipes one meter in length, joined by means of bolted flanges. They are of considerable weight, and still serve their purpose satisfactorily.

"The few repairs which have been required have generally been necessitated by the bad condition of the flange bolts, which have rusted out."*

At Clermont-Ferrand, Department of Puy-de-Dôme, on September 14th, 1746, the Commissioners of Fountains made an agreement with a Parisian iron merchant, Sieur Marchais, for furnishing about a mile of 5 or 6-in., cast-iron pipe to supply water to the fountains. This pipe was laid in 1747-48. The lower portion, in the plain, was under quite heavy pressure, and was $\frac{8}{10}$ in. thick; where the pressure was less, the thickness was only ½ in. These pipes were in 3-ft. lengths, flanged, and with a lead gasket sometimes as thick as $\frac{7}{8}$ in. In 1867, the system was paralleled by new mains, but much of the original pipe is still in use.

At Saint Etienne, in 1904, there were cast-iron pipes which had been in service since 1782, or 122 years.*

A few years ago, the City of Paris* abandoned the Chaillot Pumping Station, the first erected for supplying that city with water. The

* "The Life of Cast-Iron Pipe", by C. Cavallier, *Journal*, New England Water Works Assoc., June, 1904, p. 218.

intake, consisting of flanged cast-iron pipe extending out into the Seine, was removed after having been immersed in the river for more than a century. When removed the pipes were found to be so well preserved that they could readily have been used again, except for the fact that they were of an antique and obsolete pattern. After removing the scale and shellfish coating from the exterior, the inscription, "CREUSOT, AN 10", in raised letters, could be readily seen. This date corresponds to the year 1802 in the present calendar.

The metal was a gray, cast iron of excellent quality. No alterations of either the interior or exterior appeared after its 100 years of service, and the fractures were as sharp as those from a fresh casting.

London and Glasgow have a record of 120 years of service for cast-iron pipe. The first extensive works in the former city were built by Sir Hugh Middleton between 1609 and 1613. As early as 1746, the Chelsea Water Company, of London, laid a 12-in. flanged, cast-iron pipe, which was relaid in 1791 on account of the joints "being perished." It was the engineer of that water company, Mr. Thomas Simpson, who designed the first bell and spigot pipes. This was about 1785, at which time an experimental section with lead joints was laid.*

As a rule, however, London was supplied with water through wooden pipes until about 1820.

In the United States, the earliest pipes were of bored logs. Philadelphia adopted them, although, in 1804, a length of about ¾ mile of iron pipe was laid as an experiment. The new Fairmount Water-Works in that city had a distribution system of cast-iron pipes, which was the first in America, at all events on an extensive scale.

These pipes were imported from England, and were of the bell and spigot type, in 9-ft. lengths. A curious feature about them was that each length had three or four ribs or rings about it, approximately 1½ in. wide, which were presumably to afford places with heavier walls in which to tap the pipe.† These pipes are of particular interest owing to the fact that they are still in use after 95 years, with every evidence of continuing to give a good account of them-

* "The Development of Water Supplies and Water-Supply Engineering," by Frederic P. Stearns, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LVI, p. 455.

† Information regarding Philadelphia water pipes was furnished by the Bureau of Water of that city.

selves. Though forming only an infinitesimal portion of the present distribution system, they, nevertheless, constitute quite a considerable mileage.

A few of these pipes are noted in order to give an idea of their character: A 4½-in. cast-iron pipe on Chestnut Street, from Broad to 15th Street, laid in 1817, was removed only recently. A 22-in., cast-iron main leads from Fairmount Basin to Green Street, to Pennsylvania Avenue, to Callowhill Street. It then reduces to a 20-in. main leading to Broad Street and thence to Chestnut Street. This pipe was laid in 1819, and, with the exception of a portion which was removed in 1897 on account of the construction of the Pennsylvania Avenue Subway, is still in use. Mains on Market Street, from Juniper to Broad Streets, laid in 1820, from Eleventh Street to Juniper Street, laid in 1823, and from Water Street to Eleventh Street, laid in 1822, were in good condition when removed in 1907 to make way for the Market Street Subway. Mains, 6 and 10 in. in diameter, on Second Street, between Greenwich Street and Lehigh Avenue, were laid between 1823 and 1852; 8-in. mains on Front Street, between Wharton Street and Laurel Street, were laid at various dates between 1822 and 1834; one 10-in. main on Chestnut Street, from Front Street to Broad Street, was laid in 1821 and 1823; and a 20-in. main on Fifteenth Street, from Callowhill Street to Chestnut Street, was laid in 1829. All these mains are still in use.

This list might be extended to several pages, but it serves to show that a century's use has not rendered these pipes unfit for service. Virtually the whole of the old distribution system laid at that time has given an equally good account of itself.

The late James B. Francis,* Past-President, Am. Soc. C. E., cited the case of Lowell, Mass., as follows:

"It is within my experience and knowledge that pipes at Lowell, which were obtained from Philadelphia in 1830, were then intended to be used under a pressure of 150 feet head, and are now in use under 200 feet head, without any signs of failure after sixty years' use."

Even when immersed in the fresh water of streams and rivers, this resistance to corrosion is manifested. Cast-iron cannon taken

* *Transactions*, Am. Soc. C. E., Vol. XXIV, p. 261.

from a vessel sunk in the Delaware River for more than 40 years were perfectly free from rust.

Summing up what has been observed, it may be said that our experience of 250 years with cast-iron pipe has not been sufficiently long to establish just what its life is.

Examples of Deterioration of Cast-Iron Pipe.

A knowledge of this fact, however, may lead to very mistaken conclusions. One of the earliest observations relating to the rapid deterioration of cast-iron pipe was that of Mr. Thomas Duncan, who, in 1853, in describing the Liverpool Corporation Water-Works, said:

"During the process of taking up and adjusting the pipes in the lower districts, near the margin of the docks, where the soil is impregnated with muriate [common salt] they were found, in many instances, to have become so soft, that they could easily be cut by a knife. This was found to be the case where they had not been laid for more than twenty years; whereas, in the higher districts, where they had been laid for nearly fifty years, they were found to be as good as when first laid down. In all instances the hardest pipes had deteriorated least, and were found cleanest on the inside."*

In other words, under certain conditions, cast-iron pipe is not durable, unless protected. Therefore, it is of great importance to the engineer to determine just what those conditions are, and how to guard against them.

The salt of the ocean is particularly destructive to cast iron, and its action is manifested in a peculiar way, differing materially from ordinary rusting, scaling, and pitting.

In contrast to the case in which cast-iron cannon removed from the fresh waters of the Delaware River were found to be free from rust, Major-General Pasley† stated that the cannon of the *Royal George,* which had been sunk in the ocean for 62 years, and of the *Edgar,* which had been similarly submerged for 133 years, had been examined by him. The cast iron had generally become so soft that the metal, which resembled plumbago in appearance, could be readily cut with a knife. The cast-iron shot became red hot on exposure to air, and fell to pieces. The cannon hardened after a time, and the late William J. McAlpine,‡ Past-President, Am. Soc. C. E., said

* *Minutes of Proceedings*, Inst. C. E., Vol. XII, p. 487.
† *Minutes of Proceedings*, Inst. C. E., Vol. III, p. 86.
‡ *Transactions*, Am. Soc. C. E., Vol. I, p. 23.

that they were again fired. Be that as it may, the cast-iron guns from some ancient pirate ship were brought up from the ocean depths off Holyhead in 1822, after the lapse of a century. They were quite soft, but hardened so much on exposure to the air that they were used to fire salutes to King George IV, when he passed through Holyhead on his way to Dublin.* These old guns were said to have given louder reports than any others!

The chemist, Berzelius,† also refers to the curious heating of cannon balls, and states that some which were raised at Karlskrona, from a ship that had been sunk for 50 years, were found to have been converted into a porous, graphitic mass which heated spontaneously on exposure to air for 15 min.

It is well known that iron filings will burn in the flame of a candle, and that very finely divided iron, obtained by the reduction of its oxides in hydrogen, is pyrophorous, that is to say, it ignites spontaneously on exposure to air. These cannon balls, therefore, may have contained a remnant of metallic iron in an exceedingly fine state of subdivision. The heating may also have been due to the rapid change from ferrous to ferric hydrate on exposure to air, or to a combination of the two causes. Just why the shot should heat up and fall to pieces, when the cannon themselves did not, is difficult to explain.

The corrosive action of sea water is not uniform under all conditions, but may be materially accelerated or retarded. A case of this sort is cited by Mr. Robert Mallet‡:

"Iron under certain circumstances is subject to a peculiar increase of corrosive action—as, for instance, cast-iron piling at the mouth of tidal rivers—from the following cause. The salt water being of greater density than the fresh, forms at certain times of tide an under current, while the upper or surface water is fresh; these two strata of different constitution coming in contact with the metal, a voltaic pile of one solid and two fluid elements is formed; one portion of the metal will be in a positive state of electrical action with respect to the other, and the corrosive action on the former portion is augmented. The lower end of an iron pile, for instance, under the circumstances just mentioned, will be positive with respect to the other,

* Rennie, *Minutes of Proceedings*, Inst. C. E., Vol. IV, p. 333.
† Traité de Chimie, 1831, Vol. III.
‡ *Minutes of Proceedings*, Inst. C. E., 1840, p. 71.

and the corrosion of the lower part will be augmented by the negative state of the upper portion, while the upper will be *itself* preserved in the same proportion."

This statement would seem to be borne out by observations on the Brandywine Lighthouse,* situated on a shoal of that name in Delaware Bay, 8 miles inside the Capes. It is founded on nine, hammered-iron, screw-piles, surrounded by fifty-two rolled-iron, fender-piles. It was built in 1849-50, in about 6 ft. of water, but at times the scour and shifting sands have varied this depth materially. In 1873, an examination by a diver revealed the fact that the most serious corrosion was below the surface of low water, beyond the action of the atmosphere. In 1879, one of the fender-piles, together with a portion of the cast-iron socket, was broken off by ice. The outer $\frac{1}{8}$ in. of this casting consisted

"of a soft dark gray coating of metamorphosed iron, which could be removed readily with the finger-nail, but the piece retained the sharp outlines of its original form. The coating became hard after exposure to the air and did not separate from the iron, but the iron seemed rather to have become softened by the action of the water on its surface. * * *

"No greater deterioration, where the wrought and cast iron were in contact, was observable."

Cases occur, and with considerable frequency, where trouble has resulted from laying cast-iron mains in saline soils.

In 1902, the Elizabethtown Water Company, Elizabeth, N. J., laid about 2 miles of 6-in., cast-iron main across salt meadows. It was ordinary coated water pipe, about 0.45 in. thick, and was laid about 8 in. below the surface of a salt marsh.† The surface of the marsh was about 18 in. above ordinary high water, and, therefore, was covered at times by salt water. The nearest trolley is about 3 miles away, so that stray currents have nothing to do with the observed deterioration of the pipe. In 1909, or only 7 years after it was laid, the pipe began breaking, under a pressure of 40 lb., and it became necessary to renew the whole line in 1912, after a life of only 10 years.

Internally, the pipe was in good condition. The external appearance was equally good, there being an entire absence of pitting. On

* *Proceedings*, Engrs. Club of Philadelphia, 1884, p. 129.

† Information furnished by Mr. D. H. Townley, Superintendent, Elizabethtown Water Company.

closer examination this appearance was found to be deceptive, for, in many places, sometimes the top, sometimes the bottom, and, at other times, the sides, were found to have large spots where the iron appeared to be leached out, so that it could be cut like putty, or rather like soft graphite, which it strongly resembled in appearance. This action was not regular, sometimes skipping several feet, but was worst at the bottom of the pipe. In some instances, these soft places had been eaten completely through the walls of the pipe, which, although not pitted or swelled out, had entirely lost all the characteristics of iron. After exposure to the air, this metamorphosed iron hardened, just like the cannon used in saluting King George IV at Holyhead.

At Perth Amboy, N. J., similar trouble was experienced. A 16-in. main, crossing a salt meadow for some 3 500 ft., had to be abandoned altogether in less than 20 years. Mr. A. H. Crowell, Superintendent of the Water-Works, under date of March 20th, 1913, says:

"Our second main, a 24-in., laid under the same conditions [as the 16-in. main] about seven years ago, now shows evidence of the same deterioration, softening in spots so that it may be pared with a knife on the exterior. This line the Board has decided to uncover and leave exposed in open trench to the flow of the tides, our experience having shown that pipe when so laid exposed to the light and air is not affected; in other words, we have reached the conclusion that this particular deterioration occurs from chemical action due to contact of roots of salt grasses.

"In connection with the deterioration, one peculiarity noted is that when such pipe is removed and exposed to the light and air for a period of several weeks it will harden and ring apparently sound."

This pipe, which was laid in black meadow muck from which salt grass is cut, is buried with 3 ft. of cover. The meadow is completely submerged at extreme tides, with an ordinary range of about 4 ft. The pipe was ordinary gray cast iron, tar-coated, weighing 140 lb. per ft., and the deterioration occurred at all points, but principally on the top and upper sides. As in the Elizabeth pipe, the exterior did not pit, but deterioration occurred from softening in spots which finally blew out.

At Atlantic City, N. J.,* the water supply is delivered to the city from the Absecon Pumping Station, the mains leading through salt

*Information furnished by Mr. Lincoln Van Gilder, Engineer and Superintendent, Water Department.

marsh for a distance of 22 000 ft. In 1882, a 12-in. cast-iron main was laid, which, in 1888, was paralleled by a 20-in., cast-iron main; and, in 1901, the capacity was further augmented by laying a 30-in. steel main alongside the others. In 1911, a 48-in., wood stave main was laid parallel to the preceding three for about 7 000 ft., after which it takes a different route for the remainder of the line. Both the cast-iron mains are soft in spots, and blow out from time to time in places, under a normal pressure of 50 lb. The deterioration is precisely like that noted at Elizabeth and at Perth Amboy. The steel pipe is even less satisfactory. Though laid at a later date than the two cast-iron mains, it is rapidly deteriorating, and 6 000 ft. of it are now out of service.

The top of each of the two cast-iron mains is about 1 ft. below the surface of the meadow; the steel main is about half its diameter below the surface, and the top is covered with meadow sod.

At Richmond, Va.,* about 400 ft. of 6-in., cast-iron pipe, laid in a salt blue marl, in 20 years, became soft enough to cut with a knife, the action appearing to be equally great at its top, bottom, and sides. A trolley track, 450 ft. distant, may possibly have contributed to the disintegration.

Trouble has been experienced also from saline soils at points remote from the ocean. At Syracuse, N. Y., there are salt springs which have been used in the manufacture of salt since their discovery by the Jesuits in 1654. They are situated along the shores of Onondaga Lake, and, in 1797, were taken over by the State, which passed laws governing the manufacture of salt.

The Bureau of Water† of Syracuse has had much trouble from the corrosion and disintegration of water pipe in the salt lands. A line of pipe runs in a very low section of these lands, the major part of which is under water a large portion of the year. The part which is continually submerged, however, has not disintegrated, but that adjacent to it, where the main was laid about 1904, began to fail after several years' service. The corrosive action appears to have been similar to that noted heretofore. The pipe was sound in parts, and at the leaks it could be cut with a knife. The salt has been for a century at the point where the disintegration occurs.

* Information furnished by Mr. E. E. Davis, Superintendent, Water-Works.
† Information furnished by Mr. George A. Glynn, Superintendent.

A third soil destructive to cast-iron pipes is also noted by Mr. Glynn as occurring at Syracuse. He says, in a letter:

"Since submitting this matter to you and Dr. Pattee, I have observed two cases of disintegration in differents parts of the town, the pipe being eaten precisely as that in Spring Street. In each case the soil above the pipe was composed in the main of coal ashes and apparently the constant trickle of water had created some chemical action which ate the pipe. As a layman, I have concluded that coal ashes, where there is a possibility of a flow or trickle of water through them, are responsible for this particular kind of disintegration."

A. A. Reimer,* Assoc. M. Am. Soc. C. E., notes an instance in his experience where an 8-in. main, laid in clay and ashes—marl with some cinders mixed with it—went out of service every 6 months.

In Germany, R. Krzizan† cites a case where a portion of an asphaltum-coated, cast-iron, water main, which had been in service for 20 years, suddenly became defective. A number of conical holes were scattered irregularly over the surface and were surrounded by graphite-like material containing particles of metallic iron. The composition of this corroded material is given in Table 1.

TABLE 1.—ANALYSIS OF CORRODED GERMAN WATER PIPE.

Metallic iron	5.98
Ferric oxide and hydroxide	34.09
Ferrous phosphate	9.63
Ferrous silicate	37.16
Ferrous sulphate	0.47
Carbon	11.42
Sulphur	0.097
Manganese	1.362
Copper	0.296
	100.505

There was nothing in the water running through the pipe to account for this formation, but crystals of gypsum were distributed irregularly in the clay in which the pipe was laid. Krzizan attributes the corrosion to local currents set up by contact between the graphite particles and the iron, in the presence of a solution of calcium sulphate at the points where the crystals of gypsum were contained in the clay adjacent to the pipe.

* *Proceedings*, Am. Water Works Assoc., 1910, p. 263.

† *Zeitschrift* für öffentliche Chemie, Vol. XVIII, p. 433.

A question arises as to what effect the alkali soils of the West would have on cast-iron pipe, and it is hoped that information on this point may be brought out in the discussion of this paper. In the effort to obtain some facts bearing on this phase of the subject, the soil survey of the Uncompahgre Valley, in Colorado, made by Messrs. Nelson and Kolbe for the United States Department of Agriculture in 1910, indicated that Montrose, on the banks of the Uncompahgre River, had areas of such high alkalinity that experience there would be useful in giving an idea of what might be expected in such cases.

"The source of most of the alkali of this area is the underlying shales, and wherever they are near the surface its presence is assured."*

The low valley soils are also in general heavily charged with alkali, the principal salts being sodium sulphate (Na_2SO_4); sodium chloride (NaCl), or common salt; calcium chloride ($CaCl_2$); magnesium chloride ($MgCl_2$); magnesium sulphate ($MgSO_4$); and sodium carbonate (Na_2CO_3), named in the order of the quantities in which they occur. An analysis of a typical alkali crust found on the surface of the soil is given in Table 2.*

TABLE 2.—ANALYSIS OF ALKALI CRUST.

Constituent.	Parts per 100 000.	Constituent.	Parts per 100 000.
Calcium (Ca)	Trace.	Chlorides (Cl)	2 280
Magnesium (Mg)	.....	Sulphates (SO_4)	8 320
Sodium (Na)	2 870	Bicarbonates (HCO_3)	2 120
Potassium (K)	250	Carbonates (CO_3)	1 160
			17 110

The greater part of Montrose is underlaid by soil having an alkali content of from 200 to 400 parts per 100 000. There is a small area at its southwest corner where the alkali runs up to 600 parts per 100 000, and another area in the northwest portion where it ranges from 600 to 1 000 parts per 100 000.

Mr. R. L. Smith, Superintendent of Water Supply, states that the system, which was built in 1888, consisted of 4, 6, and 8-in., wrought-iron pipe, which deteriorated so rapidly that it was all replaced either by cast-iron or wood stave pipe, except about 800 ft.

* "Soil Survey of the Uncompahgre Valley Area, Colorado," Nelson and Kolbe, Washington, 1912.

of the 8-in. pipe in the southwestern part, which is still in very fair condition. Cast-iron pipe which has been in use for nearly 20 years has appeared to be in an excellent state of preservation, wherever it has been examined, and has never given any trouble. The alkali soil, however, has been disastrous to the wire of the spiral-wound wood stave pipe. It was laid in 1905 and will have to be removed within the next couple of years. There are no trolleys, and the pipe is laid 4 ft. below the surface.

Before attempting to draw any conclusions from the facts presented, it will be desirable to consider first the composition and character of cast iron, and second, to review as briefly as possible the theory of its corrosion, and study some experimental data bearing on the subject.

The Composition of Cast Iron.

Cast iron is a substance of varied composition, both physically and chemically, so that some conception of its formation is desirable in considering its corrosion.

It is well known that common salt added to water progressively lowers its freezing point, until, with 23.5% of salt present, the whole mass freezes at —22° cent. This is said to be the "eutectic" composition, a term which is used of a compound substance that has a lower fusing point (or freezing point) than its components themselves. The freezing point is raised if either more or less salt is present.

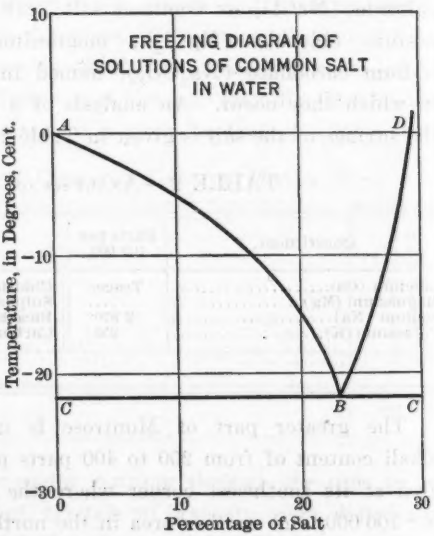


Fig. 3.

The whole is shown diagrammatically by Fig. 3.* In percentages less than the eutectic, crystals of ice form when the temperature falls to the points shown on the curve, *A-B*, the mother liquor be-

* "Cast Iron in the Light of Recent Research," Hatfield.

coming more concentrated, until the eutectic composition is reached, at which time the whole solidifies. This always occurs at a temperature of — 22° cent. If the original solution contains more than 23.5% of salt, the solution progressively freezes at the temperatures shown on the curve, *B-D*, pure salt being deposited and the mother liquor becoming less concentrated, until at — 22° cent., it diminishes in saline content to the eutectic composition, when, again, simultaneous freezing takes place. The line, *C-C*, shows the freezing point of the mother liquor. Under the microscope, it is seen that the ice is not homogeneous, but consists of innumerable plates of salt and ice in mechanical contact.

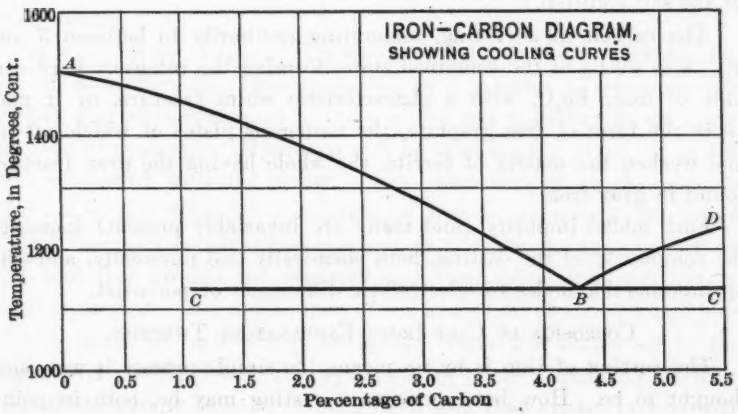


FIG. 4.

Pig iron, similarly, may be termed a frozen solution of carbon and iron. If no other impurities were present, it would contain 4.3% of carbon. Fig. 4* shows graphically how the addition of carbon lowers the solidifying point to *B*, or the eutectic, and then raises it to *D* as additional percentages are present, the line, *C-C*, indicating the constant solidifying point of the mother liquor at 1 135° cent. The free graphite found in cast iron, therefore, is due to the same action as the solid plates of salt frozen out of the water, with this important difference, that whereas ice does not retain any salt in solid solution, the solidified iron retains much carbon.

It has been known for a long time that in pig iron which has been heated considerably above its fusing point, the carbon tends

* "Cast Iron in the Light of Recent Research," Hatfield.

to separate in the graphitic form, whereas the same metal, heated to a lower degree, on cooling, would retain more carbon in the combined condition. Much study has been devoted to this obscure subject, and a series of modifications of iron and iron-carbon, all due to different conditions of heating and cooling, have been investigated, but it would be out of place to consider the subject in this paper.

To summarize briefly: the metal collecting in the bottom of a blast furnace is a saturated solution of carbon in iron, and as it cools to the solidifying point, carbide is thrown out of solution. This splits up into iron and carbon. Iron solidifies, the mother liquor splits up, and the same process is repeated. It is analagous to the behavior of the salt solution.

The carbon in a casting, amounting ordinarily to between 3 and 4%, may all be in the combined state, forming the intensely hard carbide of iron, Fe_3C, with a characteristic white fracture, or it may be in the form of free graphite, the numerous plates of which cut up and weaken the matrix of ferrite, the whole having the gray fracture found in gray iron.

Each added impurity (and many are invariably present) increases the complexity of the casting, both chemically and physically, and sets up innumerable nodes for electrolytic differences of potential.

Corrosion of Cast Iron: Explanatory Theories.

The rusting of iron is by no means the simple process it was once thought to be. How heterogeneous a casting may be, both in point of composition and in physical structure, has been shown. This adds materially to the difficulty, and explains the often apparently discordant results obtained by careful investigators.

Five separate theories have been advanced to account for the rusting of iron, but, at present, three of these have been practically eliminated, leaving only two over which the fight still wages: the acid theory and the electrolytic theory. The one undisputed fact is that iron cannot rust in air unless water is present, nor can it rust in pure water unless oxygen is present.

Mr. J. Newton Friend* has recently collated much scattered material, and has placed virtually all the researches hitherto made on this subject in the hands of the investigator. To his work, the writer is greatly indebted.

* "The Corrosion of Iron and Steel."

The Acid Theory.—This theory regards corrosion as resulting from an acid, usually carbonic, which unites with the iron to form ferrous carbonate ($FeCO_3$) or the soluble ferrous hydrogen carbonate ($FeH_2[CO_3]_2$), and the liberated hydrogen combines with any dissolved oxygen in the water to yield water. The oxygen of the air next converts the soluble iron salt into hydrated oxide, or rust, liberating the carbon dioxide (CO_2) which is now free to attack a fresh portion of iron.* With a supply of water and oxygen, a small portion of carbon dioxide could thus rust an indefinite quantity of iron.

The Electrolytic Theory.—This theory holds that the presence of an acid is not necessary to cause rusting, but regards the action as electrochemical.†

To state briefly a few of the fundamental principles, consider the case of two platinum plates immersed in a solution of common salt (NaCl), one plate connected to the positive and the other to the negative pole of a battery. Hydrogen will be immediately given off at the plate connected with the negative pole, and oxygen at the positive pole. If a few drops of a solution of litmus are added, the liquid around the positive plate becomes red, indicating the presence of an acid, and that around the negative pole turns blue, showing that an alkali is present. The appearance of the gases and of the red and blue color is noted at once when the battery wires are connected to the plates, even if the latter are separated by quite a quantity of solution, giving strong support to the supposition that matter travels with the current, part toward the positive and part toward the negative pole. Farraday named these moving portions of the solution "ions." A solution or a fused salt which conducts the electric current is termed an electrolyte.

If a steel knife-blade is immersed in a solution of copper sulphate ($CuSO_4$), iron dissolves and copper is deposited, the electric charge being transferred from the ions of copper to those of iron. Electrochemically, hydrogen is a metal, and is classed with copper in relation to iron. If, therefore, iron is placed in a solution containing hydrogen ions, a similar reaction will take place, iron being dissolved and hydrogen passing from the electrically charged or ionic state to the atomic

* Crum Brown, *Journal*, Iron and Steel Inst., 1888, No. II, p. 129.

† Whitney, *Journal*, Am. Chem. Soc., 1903, Vol. XXV, p. 394; Cushman, "The Corrosion of Iron," Public Roads Bulletin No. 30, Washington, 1907.

or gaseous condition,* and being deposited on the iron blade. Ferrous salts, as well as ferrous hydroxide, are rapidly oxidized to the ferric state, if air or oxygen is present.

It is a matter of common observation that, when iron rusts, it does not do so uniformly, but in patches and spots. Allerton S. Cushman, Assoc. Am. Soc. C. E., noticed that if this rusting took place in water or in a neutral solution of an electrolyte, such as salt, to which a little phenol-phthalein had been added, a pink color developed, and this was confined to certain nodes or spots. This pink color is a proof of the presence of hydroxyl ions, and thus indicates the negative poles. Professor Walker† suggested adding a trace of potassium ferricyanide to the solution, in order to show the positive poles. If iron goes into solution, ferrous ions must appear, which, with the ferricyanide, form Turnbull's blue. Walker further suggested stiffening the reagent with gelatine and agar-agar to preserve the effect produced. This reagent, which indicates at once the positive and negative poles, with the ferrous ions shown by blue at the positive and the hydroxyl ions by pink at the negative poles, was named ferroxyl by Mr. Cushman.

Later, the writer will show the behavior of some of the corroded water pipes when ferroxyl is applied.

Whatever may be the abstract facts concerning the electrolytic and acid theories, if absolutely pure iron, pure oxygen, and pure water are used, both reactions take place under actual conditions of corrosion.

Natural waters are usually charged with dissolved oxygen and carbon dioxide from the air, thus readily permitting the action of carbonic acid on the iron. Even the purest commercial wrought irons are not homogeneous, either physically or chemically, so that electrolytic action also comes into play.

Conditions Contributing to Corrosion.

Hale,‡ in studying experimentally the corrosion of iron pipes by water flowing through them, concluded that practically carbonic acid, coupled with dissolved oxygen, is a most potent factor in corrosion. The iron is dissolved as bicarbonate with the release of hydrogen:

$$Fe + 2H_2CO_3 = FeH_2(CO_3)_2 + H_2.$$

* Whitney, *Journal*, Am. Chem. Soc., 1903, Vol. XXV, p. 894; Cushman, "The Corrosion of Iron," Public Roads Bulletin No. 30, Washington, 1907.

† *Journal*, Am. Chem. Soc., 1907, Vol. XXIX, p. 1257.

‡ *Journal*, Am. Public Health Assoc., Vol. III, p. 1337.

About 20% of this hydrogen re-united with the dissolved oxygen in the water. The soluble iron bicarbonate was oxidized also by the dissolved oxygen to insoluble red oxide, setting free the carbonic acid:

$$4FeH_2(CO_3)_2 + O_2 + 4H_2O = 2Fe_2O_3 + 8H_2CO_3.$$

This freed carbonic acid again dissolves more iron and is again set free, until all the oxygen is exhausted. As a result of the catalytic action of the carbonic acid and water, a very large quantity of iron is corroded for each part per million of dissolved oxygen. Theoretically, 10 parts per million of dissolved oxygen produce 126 parts per million of Fe_3O_4, expressed as Fe; whereas, if the hydrogen were oxidized and not set free, the quantity oxidized by 10 parts of oxygen would be only 31 parts per million of iron. By experiment, Hale found that from 75 to 80% of the theoretical quantity of iron was actually oxidized.

Although, theoretically, the oxide is the final stage, the differing colors of red, green, brown, and black indicate that the iron exists in hydrated form, except when formed in the hot way.

Hale also found that the rate of corrosion increased proportionately to the increase of free carbonic acid. Analyses for soluble iron and dissolved oxygen proved that this increase was due to the fact that the carbonic acid dissolved the iron more rapidly, and more iron being in solution, oxidation is hastened.

The aqueous solutions of most acids dissolve iron. For example, if hydrochloric acid is used, ferrous chloride and hydrogen are the resultant products, the former oxidizing into ferric chloride in the presence of air. If only a trace of acid is present, the iron will be deposited from the solution as hydrated ferric oxide or rust.

Cast iron subjected to the slow action of dilute acids, as, for example, acid water from mines, retains unchanged the external shape of the casting, but the composition of the metal is altered, iron being removed in solution.

Very dilute solutions of alkaline hydroxides, such as sodium hydroxide (NaOH), or caustic soda, and calcium hydroxide ($Ca[OH]_2$), or slaked lime, rapidly absorb carbon dioxide, forming carbonates which in such dilute solution do not prevent the iron from rusting. On the contrary, by hastening the absorption of oxygen by the iron, they increase corrosion.

If a plate of iron is cleaned, weighed, and placed in a beaker with either water or a solution of a salt, and after remaining in the solu-

tion for some fixed period, is removed, cleaned, dried, and weighed, the loss in weight measures the quantity of corrosion.

Taking 100 as the rate of corrosion of pure iron in water, Fig. 5 shows the rates in varying strengths of salt (NaCl) and sodium sulphate ($Na_2SO_4 10H_2O$), according to the experiments of Friend and Brown.* The surprising fact is that, whereas dilute solutions of these salts increased the corrosive action of water, stronger solutions exerted less corrosive action on pure iron than water alone. This bears out the experiments of Adie, made some 69 years ago.† He took weighed pieces of wrought and cast iron; immersed some in pure water and some in saturated solutions of salt, and after the lapse of several days, re-weighed them, with the results shown in Table 3.

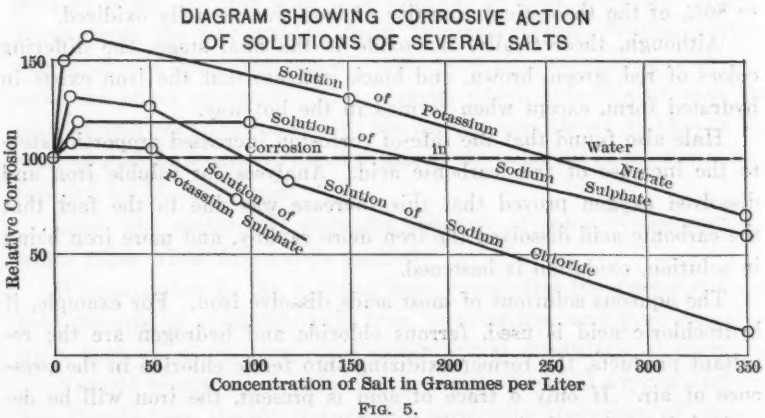


FIG. 5.

TABLE 3.

Conditions of the experiments.	Loss in fresh water, in grains.	Loss in salt water, in grains.
Twenty pieces of wrought iron, weighing 374 grains, exposed for 80 days	1.9	0.1
Three rods of cast iron, weighing 787 grains, exposed for 62 days	1.6	0.4

Both kinds of iron were corroded much more in the fresh water than in the saturated solution of salt water. Adie attributed this to the decreased solubility of oxygen in salt solutions. Friend proved experimentally that, in the absence of air, solutions of sulphates and chlorides of sodium and potassium have no action on pure iron.

* "Corrosion of Iron and Steel," by J. Newton Friend, 1911.
† *Minutes of Proceedings*, Inst. C. E., 1845, Vol. IV, p. 323.

Rain-water, particularly in thunder-storms, contains nitric acid or nitrates, and polluted waters also may contain nitrates and nitrites in solution. The curves on Fig. 5, showing the corrosive action of potassium nitrate and of potassium sulphate, also from Friend and Brown, are interesting, the more so as it is very dilute solutions of these substances which are ordinarily encountered. As these curves show, their corrosive effect is then at a maximum.

Magnesium and ammonium chlorides are found to corrode iron, even in the absence of air. This gives a clue to the particularly destructive effect of sea water (which contains magnesium chloride) on cast iron, even at great depths below the surface, where fresh water would not contain enough dissolved oxygen to exert any deleterious influence on the iron.

With mixtures of two or more electrolytes in one solution, the subject becomes so broad that a vast amount of investigation remains to be done.

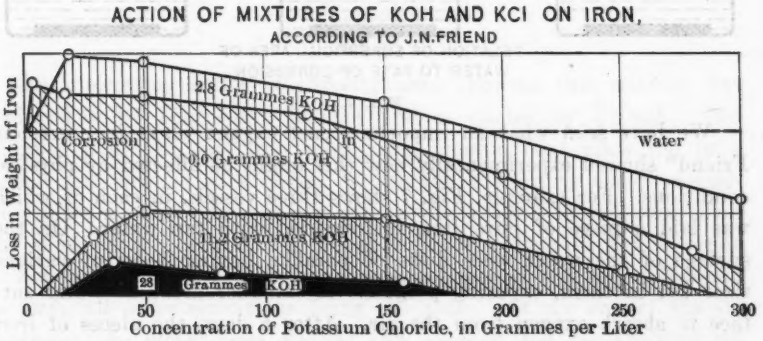
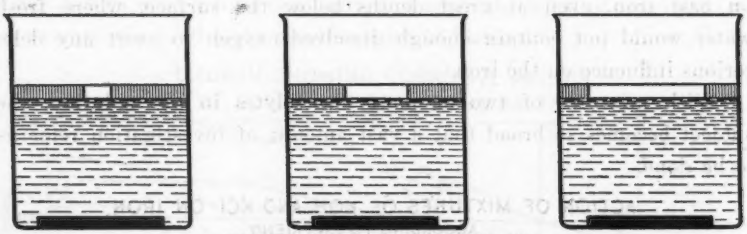


FIG. 6.

It is very important, however, to have some idea of what occurs under such circumstances. Friend* placed 36 pieces of pure iron foil in a series of hard glass beakers, the first containing water, and the others containing water in which varying quantities of potassium chloride and potassium hydroxide (caustic potash) were dissolved. These were allowed to remain 28 days, after which the iron plates were removed, cleaned, and weighed. The results are shown graphically by Fig. 6. From this it will be seen that pure water corrodes iron much more rapidly than water containing caustic potash; that

* *Journal*, Iron and Steel Inst., 1911.

the corrosive action is greatly increased by the presence of potassium chloride; that increasing quantities of chloride first increase and then gradually diminish the intensity of corrosive action; and that increasing quantities of caustic potash, after a temporary increase in dilute solution, diminish corrosive action, finally inhibiting it altogether. It is this action of alkalis which explains the protection of steel and iron in concrete, the alkalis in the cement exerting a protective action, in spite of the simultaneous presence in the cement of corrosive electrolytes such as sulphates of magnesium and alkaline earths.



RELATION OF SUPERFICIAL AREA OF WATER TO RATE OF CORROSION

FIG. 7.

We have seen what an important part oxygen plays in corrosion. Friend* showed experimentally how this might be affected by physical conditions. For example, Fig. 7 shows three vessels, in each of which was laid a piece of pure iron foil covered with tap water. On the surface of the water were disks of paraffin, having holes of various sizes cut in them, exposing progressively smaller areas of water surface to absorb oxygen from the air. After 5 days, the pieces of iron foil were weighed, and the relative results are given in Table 4.

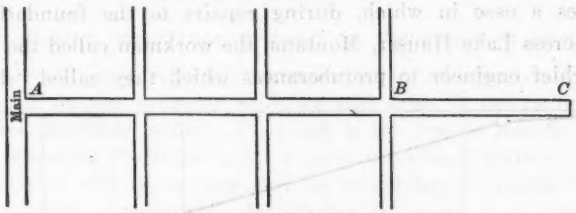
TABLE 4.—RELATION OF SUPERFICIAL AREA OF WATER TO RATE OF CORROSION.

Superficial area of water, in square centimeters.	Relative corrosion.
56.8	100
12.6	58
0.8	14

The rate of motion of water also has a bearing on the rapidity of corrosion. In certain circumstances, it causes greater aeration,

* "The Corrosion of Iron and Steel," p. 75.

and it also brings a constantly renewed supply of air to the rusting metal. Mr. M. B. Jamieson* gives an interesting example of this, illustrated in Fig. 8, in which a main, running from a larger main at *A* to a dead end at *C*, had a number of side branches taken off it. There was, therefore, a continually diminishing quantity of water and a diminished rate of flow, the greater the distance from *A*. After 45 years of use, the pipe was found almost choked with rust at *A*, but with less and less of it as the point *C* was approached. The pipe was practically clean from *B* to *C*.



WATER MAIN, SHOWING RELATION OF VELOCITY TO CORROSION

FIG. 8.

An interesting series of investigations showing this relation was undertaken by Heyn and Bauer,† and is quoted by Friend. Two weighed plates of cast iron were hung from a bent glass rod in a beaker and covered with water. A stream of running water was admitted by a glass tube and the excess was allowed to run away over the sides of the beaker, the operation being continued for a period of 3 weeks. Several beakers were used, the rate of flow being different in each, all other conditions being the same. At the end of the experiment, the plates were cleaned and weighed, the loss showing the corrosion. Fig. 9 shows the results of these experiments. It will be seen that a gentle flow increases the corrosion to more than eight times that in still water, but that it gradually diminishes as the rate of flow increases.

It is a noticeable fact that in most cases of bad pipe corrosion that have come to the writer's notice, not only was there salt water, aeration, and more or less motion in the water, but the pipes were laid, as a rule, in muck with rank swamp vegetation and life in close contact with the metal. The action of living matter, therefore, becomes

* *Minutes of Proceedings*, Inst. C. E., 1881, Vol. LXV, p. 323.

† *Mitteilungen* aus dem Königlichen Materialprüfungsamt, Berlin, 1910, Vol. XXVIII, p. 93 *et seq.*

a subject of interest, and will be found to merit careful consideration in many instances in which it has not hitherto been given a thought.

The direct action of any organism is doubtful, although it is known that organisms do occur in the tubercles forming on the inside of water mains. Dr. Brown* states that "Organisms are found in some incrustations and may modify their composition; on the other hand, incrustations are often free from every trace of life." Mr. Richard Gaines† found it probable that certain sulphur bacteria were responsible in part at least for rust tubercles and pitting in conduits. He also cites a case in which, during repairs to the foundations of a bridge across Lake Hauser, Montana, the workman called the attention of the chief engineer to protuberances which they called "shell rust"

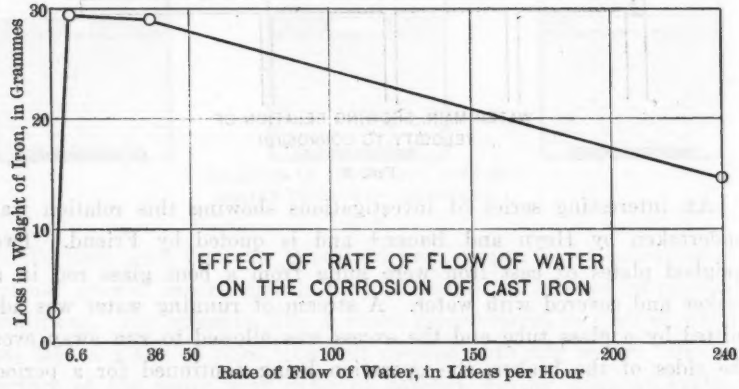


FIG. 9.

and which were found scattered over the surface of submerged steelwork. Schorler identified in this rust *Gallionella ferruginea* in large quantities. Apparently, the iron is dissolved by some acid secretion from these organisms, and, in this instance, the structural damage to this bridge which had only been in service for one year, had been enormous. It is even necessary to consider the earthworm. It has been estimated that, on an average, some 25 000 worms exist in every acre of soil. Darwin‡ concludes from his observations that each worm annually ejects 20 oz. of soil as castings, which means that 15 tons of soil in every acre of land pass through the worms each year. These castings are acid when fresh, and contain also small quantities of

* *Minutes of Proceedings*, Inst. C. E., Vol. CLVI.
† *Chemical News*, 1910, Vol. CI, p. 205.
‡ "Vegetable Mould and Earthworms."

ammonia. With 15 tons of acidified soil per acre per annum, with a small admixture of ammonium salts, there may well be a corrosive action on any embedded iron.

Darwin placed some worms in a pot of sand which, owing to the presence of ferric oxide, was red in color. The sand which passed through the worms and was found in their burrows, was bleached, and on microscopic examination, it was found that the ferric oxide was completely dissolved. Acetic acid scarcely produced any effect on this sand, yet the acid secretions of the worms decolorized it effectually.

Of still greater importance, probably, is the action of plants:

"Young roots, and especially root hairs, in addition to exhaling carbonic acid, excrete stronger acids or acid salts by means of which they dissolve insoluble bodies. That this is the case is readily demonstrated by allowing the roots to grow upon a polished plate of marble, when the latter will be so corroded as to exhibit an etched pattern showing the path of the roots. By placing the roots upon paper dyed with litmus or other sensitive indicator their acidity may readily be demonstrated. Furthermore, when plants decay, humus acids result which are powerful solvents."*

Our knowledge of the humic acids is very meager. It has been generally considered that they had an important rôle in the corrosive action of swampy waters. J. W. Ledoux,† M. Am. Soc. C. E., was troubled with the corrosive action of water at Charleston, S. C. This water was obtained from a reservoir covering 2 300 acres, and submerging a large quantity of peaty matter, much of it 15 ft. deep, a large part of the water-shed of some 49 sq. miles being low and swampy. He made a series of tests by boiling weighed pieces of piano wire in 500 cu. cm. of the water. This was done with raw water, water previously boiled to free it from oxygen and carbonic acid, and water to which various alkalis had been added. The loss in weight of the wire determined the relative corrosion. He was

"unable to determine whether ionization is the cause of the trouble, but rather believes in the theory that the raw water contains organic compounds which are corrosive in effect and with the elimination of organic matter as the result of the use of sulphate of alumina and filtration, these effects largely disappear."

* Friend, "Corrosion of Iron and Steel," p. 105.
† *Engineering Record*, Vol. LX, p. 701.

The indirect action of humus may be extensive under certain conditions. Jodidi* refers to the fact that the amount and nature of humus formed depends on the organic materials which undergo humification, on temperature, moisture, aeration of soil, presence of salts, acids, etc., and the character and quality of microbes present. The comparatively easily soluble products are leached out, and the less soluble ones accumulate in the humus. The humus is further oxidized by the action of air; the carbon to carbon dioxide; the hydrogen to water; the nitrogen to nitric acid; and the sulphur to sulphuric acid; and putrefaction leads to a number of secondary products, such as amino acids and acid amides.

How important these acids may become practically is illustrated by some recent experiences in Illinois. Clifford Older, Assoc. M. Am. Soc. C. E., in a paper† before the Illinois Society of Engineers and Surveyors, says:

"In southern Illinois a peculiar condition exists which undoubtedly has a material bearing on the corrosion of steel structures. In that portion of the State popularly known as Egypt, a large proportion of the surface soil, when tested, shows a marked acid reaction. This condition exists generally throughout the greater part of the state which lies south and east of the Kaskaskia River and south of the latitude of Mattoon. To a less marked degree, the soil is acid in many places elsewhere in the state where land has been farmed for 50 years or more.

"This acid condition does not exist for a depth of more than a few inches, and steel buried at a greater depth does not seem to exhibit any unusual tendency to corrode."

He notes the complete destruction of the webs of I-beams and channels having a thickness of ¼ in. in from 10 to 12 years.

Dr. Adeney ascribes the corrosive action of the Vartry water supply of Dublin to the presence of minute quantities of peaty matter, which ferment slowly, with the formation of carbonic acid, as well as small quantities of nitric acid.

The destructive effect of carbonic acid has already been referred to; in swampy waters, for the reasons mentioned, it is high, and, in many well waters, it is very high. It is, moreover, very soluble in

* "The Chemistry of Humus," *Journal* of the Franklin Inst., Vol. 176, p. 565.
† *Engineering News*, Vol. LXXI, p. 354.

water, so that it is in available form to attack any iron exposed to its influence.

It can be readily understood, therefore, that, by their various secretions, plants and animals are important accessories in the corrosion of iron.

Conditions Inhibiting Corrosion.

In reviewing the conditions contributing to corrosion, it was shown that whereas dilute solutions of alkalis did not retard corrosion, strong solutions neutralize the acids and preserve the iron for an indefinite period. Mr. Allerton S. Cushman found that 5% of lime applied to boggy, sour land exerted a pronounced retarding action on the corrosion of ironwork embedded in it.* The same authority further says:

"There should be many cases where the property of alkalis to inhibit corrosion could be made of more practical use than has been done. Whenever iron posts or standards are set directly in the ground, the liberal use of slaked lime should be beneficial."

The effect of concrete in protecting iron and steel was referred to when discussing the action of two or more electrolytes in one solution.

Cast iron is obtained by pouring the molten metal into sand moulds. The outer skin, therefore, is silicious. It has long been known that silicon tends to protect iron from corrosion, and Jouve† has shown that a 20% alloy of iron and silicon is not attacked by acids, but is virtually uncorrodible. Wallace‡ refers to having seen natives in India forge iron on a stone anvil, and says this iron, which was charcoal iron, does not rust on exposure to the weather, but takes on a fine brown patina. The question was raised whether the stone anvil did not siliconize the skin of the iron. In the experiments by Messrs. McCollum and Logan§, it was found that, under a given length of exposure in soil, if the corrosion of machined cast iron be considered as 100, that of cast iron still retaining the original surface as cast would be only 93. Krohnke‖ also notes the fact that the surface formed when pipes are cast, enables them to resist attack for a long time.

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* *Journal*, Iron and Steel Inst., 1909, Vol. LXXIX, p. 33.

† Friend, "Corrosion of Iron and Steel," p. 229; Jouve, *Journal*, Iron and Steel Inst., 1908, Vol. LXXVIII, p. 310.

‡ *Journal*, Iron and Steel Inst., 1908, Vol. LXXVI, p. 84.

§ "Electrolytic Corrosion of Iron in Soils," p. 31.

‖ *Gesundheits-Ingenieur*, Vol. XXXIII, p. 398.

Thwaite prepared a table (reproduced as Table 5) from various published data, showing well the protective effect of this skin.

TABLE 5.—RELATIVE CORROSION OF CAST IRON WITH AND WITHOUT "SKIN".

Metal	Foul sea water.	Clear sea water.	Foul river water.	Pure air and clear river water.	City air or sea air.
Cast iron.....	3.500	3.386	2.034	0.604	2.637
Cast iron (skin removed by planing).....	12.275	4.738	3.884	0.584	4.763

Silicon is an undesirable alloy for iron in many ways, so that a surface treatment, such as naturally occurs in casting, is probably the only method of making use of its rust-inhibiting properties.

Chromium, nickel, and copper, when present in small quantities, contribute greatly to the resistance of iron to corrosion. A steel containing about 1% of chromium was found by Hadfield to corrode about half as much as ordinary mild steel. Similarly, 3% of nickel added to steel reduced its relative corrosion in salt water as compared with open-hearth steel, about 50 per cent.*

Phosphorus in iron also retards corrosion, differing in this respect from sulphur, which, by forming sulphuric acid, powerfully attacks the metal.

The coating of iron with zinc and tin is also used to inhibit corrosion. Zinc is electro-positive to iron, and if the two metals are in contact, and are immersed in a corroding solution, the zinc will pass into solution and the iron will remain unaffected. Galvanizing, therefore, is not merely a mechanical shield to the iron, but protects the metal for some distance around it. It is well to emphasize this fact, as it is a point frequently misunderstood. In a pocket manual of tables and information for engineers, bearing the date of 1911, this statement occurs: "With both zinc and iron exposed and in contact with water, corrosion proceeds more rapidly than it would if the zinc were not present." This statement is absolutely misleading and erroneous.

Tin, on the contrary, is not electro-positive to iron. Being itself practically unaffected by corrosion, it protects the iron mechanically. The tin coating, as commercially applied, is full of minute pin holes.

* Friend, "Corrosion of Iron and Steel," pp. 217-225.

FIG. 10.—FOUNTAIN OF NEPTUNE AT VERSAILLES.

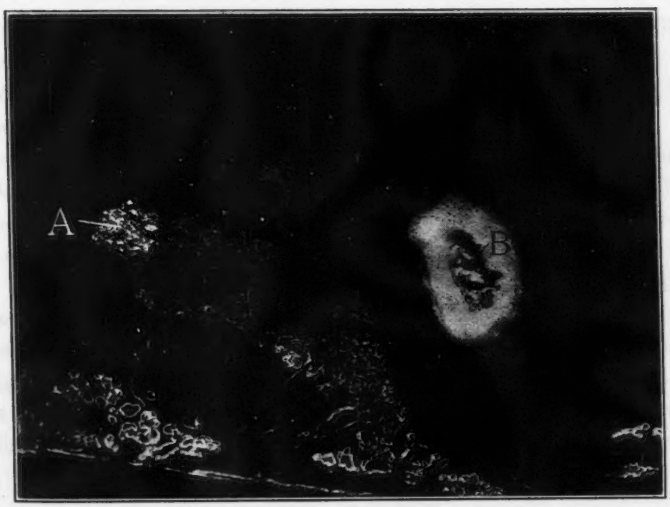


FIG. 11.—BEHAVIOR OF TIN (A) AND ZINC (B) IN RETARDING CORROSION OF IRON.

It does not inhibit corrosion of the iron at these points, thereby differing from zinc. Fig. 11 shows well the dissimilarity between the two metals in this respect. A plate of polished sheet iron was placed in a dilute solution of salt and water. At *A* was laid a lump of granulated tin and at *B*, a lump of granulated zinc. In the photograph may be seen the area of polished iron free from rust around the lump of zinc, whereas the tin had absolutely no effect in protecting the iron in its vicinity.

A Study of Typical Cases of Deterioration of Cast-Iron Pipe.

With these various facts in mind, an examination of certain typical cases of pipe failure may give information of value to the engineer who is called on to design works involving pipes subjected to deleterious influences.

It has already been noted that most, if not all, pipe failures occur from the influence of salt or saline waters.

A 10-in. main runs southward from Elizabeth, N. J., through the meadows to Morse's Creek. At this point, it is continued by a 6-in. line, also running southward. The general elevation of the meadow, as previously mentioned, is about 18 in. above mean high tide, and the bottom of the pipe is approximately at the level of mean high tide. There is no trolley near. Curiously enough, the 10-in. pipe, which was laid in 1893, has shown no appreciable deterioration except at a point opposite the works of the Standard Oil Company, where escaping acid waters caused corrosion. The 6-in. line, although not laid until 1902, has broken down completely. Both sizes were made by the Warren Foundry Company, and there is no apparent reason for the greater resisting power of the 10-in. line.

The salt marsh is intersected by creeks and ditches, and the surface is largely covered with fox grass. There are patches too wet for the fox grass, and these are covered with what is known as quill sage, a coarse, rank, swamp growth. It is in these patches that the chief damage to the pipe occurs. Fig. 12 illustrates the character of these meadows. The coarseness of the quill sage distinguishes the patches covered by it from those on which the finer fox grass is found.

Fig. 13 shows a photograph of a ring of the 6-in. pipe, and Fig. 14 shows a section of this pipe. In the ring, two places deeply corroded will be noticed at "B", and yet the exterior is smooth, to casual ex-

amination sound, and is neither swelled nor pitted, but retains its original sharp outline. At "A" the same thing is noticed, except that a slight crack occurs between the sound and corroded parts in one place. Just below "A", the corrosion extends nearly through the pipe. This condition exists in the section shown in Fig. 14, where the pipe has been completely penetrated. The interior is smooth and not tuberculated.

The corroded portion has the general appearance and consistency of graphite, and cuts like that substance. When first removed from the swamp, it is much softer, more like soft putty. When it is pulverized for analysis it is brown, and makes a brown streak. It is possible to write on paper with a fragment of the pipe, the marks resembling those made by a brown crayon.

If some filings of the unchanged metal are sifted on a card beneath which is placed a horse-shoe magnet, they arrange themselves in magnetic curves, as shown to the left of Fig. 15. Filings from the corroded part, on the contrary, if also sifted on a card with the magnet beneath, fall as an inert mass, which is shown on the right side of Fig. 15. From this it will be seen that, when carried to completion, the iron is completely changed from the metallic state, and that which is not leached out as chloride, remains as hydroxide.

One piece of the corroded portion contained only 42.2% of iron. Another portion had the composition given in Table 6, according to the analysis by Mr. Edward J. Pugh.

TABLE 6.—Corroded Portion of Pipe from Elizabeth.

Iron	50.62 per cent.
Silica	0.31 " "
Chlorine	0.72 " "
Manganese	0.46 " "
Silicon	22.32 " "
Combined carbon	0.63 " "
Graphitic carbon	5.51 " "

At a part not very deeply corroded, the specific gravity of a complete cross-section of the pipe was 5.18. The specific gravity of a piece from the corroded part of the pipe was only 2.2. The specific gravity of cast iron, as given by Trautwine, varies from 6.9 to 7.4, and is assumed to be 7.15.

Three microscopic views of a polished section of the uncorroded pipe are shown in Figs. 16, 17, and 18. Fig. 17 is magnified 50 diam-

FIG. 12.—SALT MEADOW, ELIZABETH, N. J., SHOWING COARSE PATCHES OF QUILL SAGE AMONG THE FINE FOX GRASS.

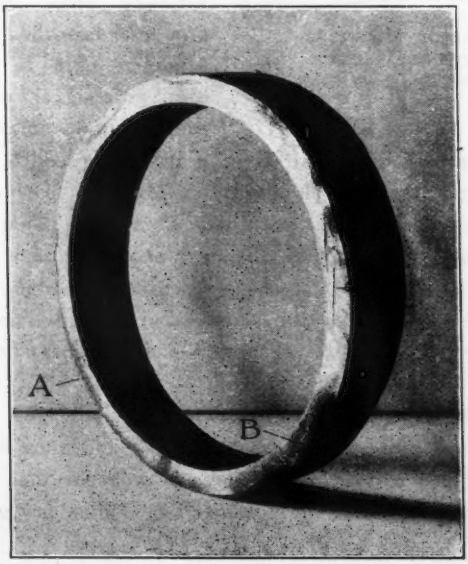
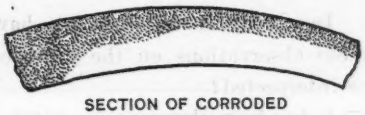


FIG. 13.—RING FROM 6-INCH CAST-IRON MAIN, ELIZABETH, N. J., SHOWING CORROSION.

eters, Fig. 18 40 diameters, and Fig. 16, approximately, 150 diameters. The thin plates of free carbon referred to in describing the composition of cast iron can readily be seen, interspersed with the ferrite. Fig. 19 shows a micrograph of a portion of the corrosion magnified 40 diameters. Owing to the softness of the material, it is impossible to obtain a high polish on these specimens, so that the view is somewhat obscured by the shadows, but, in Fig. 20, where the carbon is indicated by heavy lines, it will be seen that the graphite plates are much thicker than they are in the unchanged metal.*

Fig. 21 shows in plan and section a piece of the pipe corroded in spots. In this, as in the other examples, the sharp original outline of the metal remains even after the metallic iron has been completely removed or oxidized. A paraffin dam was placed around this piece of pipe, and ferroxyl reagent poured on. The lower part of Fig. 21 shows the result. Where there was a deep corrosion, as at the bottom of the figure, no change took place. The iron alongside this developed two red nodes, showing the presence of hydrogen ions, and a wavy cloud of blue indicates where the iron is passing into solution. A section of pipe, similar to Fig. 14, when placed in a petri dish with ferroxyl, strongly developed the red color all along the outside or deeply corroded part, and a perfect mass of blue from all the uncorroded portion. This action is much more vigorous than that undergone by either a steel wire nail, a wrought-iron nail, or a piece of cast-iron soil pipe lying in the same petri dish. The chlorine present in the corrosion (as shown by Table 6) doubtless accounts for the vigor of the reaction observed.



SECTION OF CORRODED PIPE TAKEN FROM SALT MARSH, ELIZABETH, N.J.

FIG. 14.

At Atlantic City the behavior of the pipe was similar to that at Elizabeth. The conditions also were analagous, the chief difference being that the salt meadow was only 6 in. above mean high tide, so that the pipe was subjected to more frequent alternations of water and moist air.

Two samples of soil were collected by the writer from the Atlantic City meadows, one of them ½ in. from a corroded part of the cast-iron

* See page 1651, comments on analysis of corroded pipe at Syracuse.

main, and the other 3 ft. away. The analysis made by Mr. Edward J. Pugh, is given in Table 7.

TABLE 7.—Analysis of Soil from Atlantic City Meadows.

	Soil within ½ in. of pipe.	Soil away from pipe.
SiO_2 (insol. in HCl)	18.68	32.27
Al_2O_3, CaO, etc.	10.57	22.78
SiO_2 (sol. in HCl)	0.23	0.28
Al_2O_3 " " "	1.91	1.19
Fe_2O_3 " " "	46.57	9.11
NaCl	1.93	0.95
CaO (sol. in HCl)	0.99	0.83
MgO " " "	0.00	0.00
Sulphates	0.00	0.00
Organic acids	0.00	0.00
H_2O (at 105° cent.)	6.28	28.92
Volatile matter besides H_2O (ignition)	13.27	3.96
	100.43	100.29

In the light of what we have learned experimentally, what do these observations on the corroded pipe signify, and how shall they be interpreted?

It has been shown (page 1658) that aqueous solutions of most acids dissolve iron. If hydrochloric acid is used, ferrous chloride and hydrogen are the resultant products, the former oxidizing into ferric chloride in the presence of air. It has also been shown that, if only a trace of acid is present, the iron will be deposited from the solution as hydrated ferric oxide. It is probable* that in salt water the reaction may be represented by the equation:

$$Fe + 2NaCl + 2H_2O = FeCl_2 + 2NaOH + H_2.$$

In the presence of air or of dissolved oxygen in the water, this free hydrogen oxidizes to water, the iron in solution yields ferric hydroxide, and the process continues. The chemical analysis of the portion practically free from metallic iron contains as low as 42.2% of that element, a small part of which is in the form of chloride. Clearly, part of the iron has been removed in solution, and part remains as the hydroxide, the carbon present in the combined form being left behind, and, as shown in the microscopic sections, has thickened the plates of free carbon originally in the free iron.

* Friend, "Corrosion of Iron and Steel," p. 147.

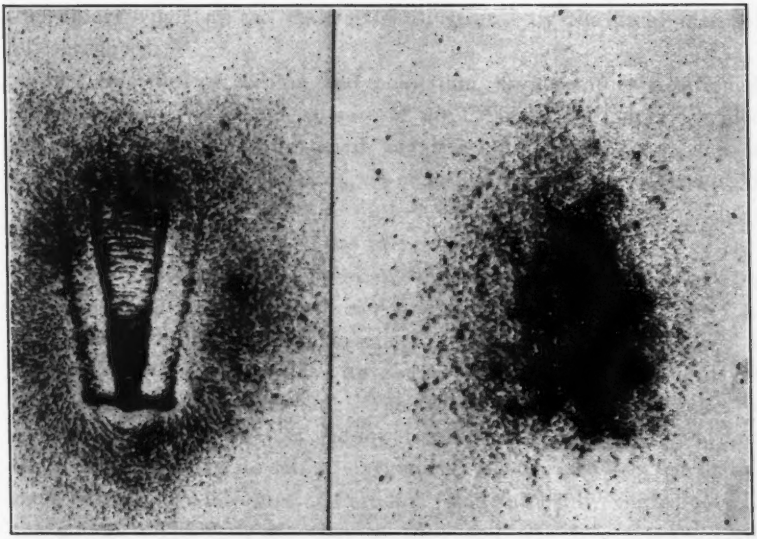


FIG. 15.—BEHAVIOR TOWARD THE MAGNET OF FILINGS FROM CORRODED AND UNCORRODED PARTS OF PIPE COMPARED.

FIG. 16.—SECTION OF CAST-IRON PIPE, ELIZABETH, N. J., MAGNIFIED 150 DIAMETERS.

The oxides of iron are electro-negative to the metal, so that electrolysis plays its part, as shown by the ferroxyl reagent, and the extraordinary vigor of the reaction is illustrated by the great mass of blue produced.

The chlorides of sodium and potassium, as heretofore mentioned, cannot corrode iron in the absence of air. Magnesium chloride, which is present in sea water in small quantities, is able to do so, but to get the full corrosive effect of sea water, plenty of air is essential.

It has been shown (Fig. 7) that the greater the surface of liquid exposed to the air, the more the corrosion is intensified.

In the salt marshes in question, the tide rises and falls, passing through a multitude of creeks and ditches all over the meadows. At Elizabeth, it ordinarily reaches about to the bottom of the pipe; at Atlantic City, it covers it. As shown in Fig. 22, this affords an ideal condition for corrosion. The liquid at high tide touches or surrounds the pipe, as in *A*; at low tide, it falls, air rushes in to take its place, and a moist, saline, oxygenated, spongy mass of earth and roots surrounds the pipe. Carbonic acid probably contributes to the action. The Atlantic City meadow soil was free from organic acids. In the sample of soil taken ½ in. from the pipe, there was 46.57% of Fe_2O_3 and even 3 ft. away, iron occurred in large quantities, there being 9.11% of Fe_2O_3. This would appear to show that it had been dissolved from the pipe as $FeCl_2$, and, after diffusing or flowing away from the pipe, as the tides rose and fell, was deposited as hydroxide.

It is further to be noted that at Elizabeth, where, on account of the elevation of the meadow, the top of the pipe was not so frequently wet as the bottom, the blow-outs generally were in the bottom of the pipe. At Perth Amboy, the top and upper sides suffered most. At Atlantic City, also, the condition of the pipe was worse at the top than at the bottom. This was to be anticipated on account of the relative elevations of the pipe and high tide. At Elizabeth, the high tide reached the bottom of the main; consequently, the top was not subjected to the saline influences so frequently. At Atlantic City, the pipes, ordinarily, were covered at high tide, and, being laid farther below the surface, air was more plentiful at the top, which, therefore, in presence of an ample quantity of salt water and with a more abundant supply of oxygen than the bottom, suffered most.

At Syracuse, the portions constantly submerged did not give out. Probably, too, the concentrated saline content in these brine-soaked lands produced an inhibiting effect, as shown in Fig. 3. The damage took place above the permanent water table, where both air and salt water were enabled to reach the pipe.

Analyses of the corroded and uncorroded portions of the Syracuse pipe were made for the Water Department by Dr. Ernest N. Pattee, of Syracuse University, and are given in Table 8.

TABLE 8.—ANALYSES OF CORRODED PIPE FROM SYRACUSE, N. Y.

	Non-corroded portion.	Corroded portion.
Silicon	0.15 per cent.	1.49 per cent.
Sulphur	0.11 " "	0.20 " "
Phosphorus	1.26 " "	1.39 " "
Manganese	0.30 " "	0.38 " "
Total carbon	3.65 " "	4.21 " "
Graphitic carbon	2.85 " "	3.84 " "

"The corrosion contained 33% of insoluble residue consisting of about 10% graphitic carbon and 22% of silicon (Si) and silica (SiO_2). Total carbon in the corrosion = 11.2%, consisting almost entirely of graphitic carbon. Sulphur in corrosion = 3.0%. Apparently, the process which accounts for the corrosion either dissolves the greater part of the combined carbon or converts it into the graphitic variety."

An analysis of the soil surrounding this pipe, made by Mr. Edward J. Pugh, disclosed that it was acid to phenol-phthalein, and contained 3.42% of organic acids. This is interesting, from its analogy to the soil conditions in Illinois, cited by Mr. Older, where steel bridges were damaged so badly by the acid soil.

On the whole, the evidence chiefly observed with cast-iron pipe would seem to indicate that organisms, whether animal or vegetable, had but little to do with the effects observed. Salt water and air seem to be the active agents, probably aided by carbonic acid, and the exclusion of either is followed by increased durability of the pipe.

The evidence given by Mr. Reimer, in which cinders caused pipe corrosion, the similar experience at Syracuse, and the German case resulting presumably from local electrolysis due to the presence of gypsum crystals, indicate the necessity for caution where pipe is likely to be subjected to such influences. Some years ago, a tunnel leading from the Susquehanna River to the power-house of the Cen-

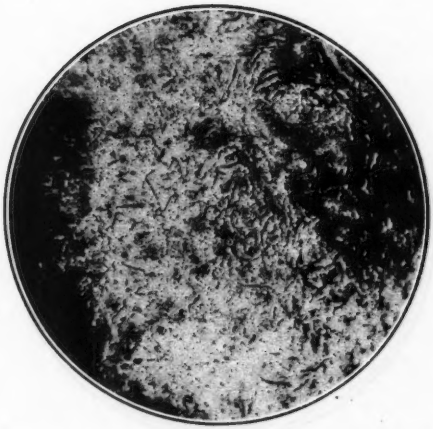


FIG. 17.—SECTION OF CAST-IRON PIPE, ELIZABETH, N. J., MAGNIFIED 50 DIAMETERS.

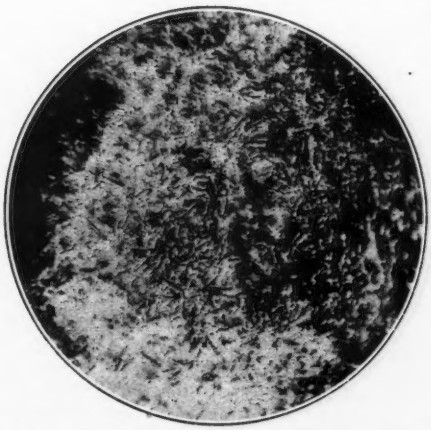
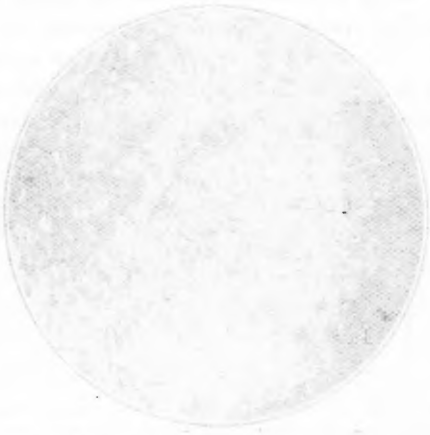


FIG. 18.—SECTION OF CAST-IRON PIPE FROM ELIZABETH, N. J., MAGNIFIED 40 DIAMETERS.



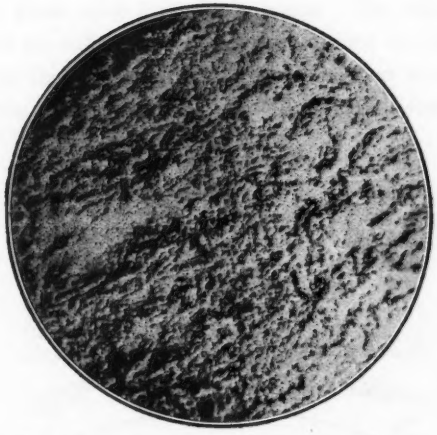
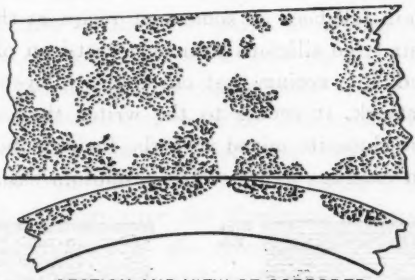


FIG. 19.—SECTION OF CORRODED PORTION OF PIPE FROM ELIZABETH, N. J., MAGNIFIED 40 DIAMETERS.

FIG. 20.—SECTION OF CORRODED PORTION OF PIPE FROM ELIZABETH, N. J., MAGNIFIED 40 DIAMETERS, SHOWING CARBON PLATES.

tral Pennsylvania Traction Company, at Harrisburg, was constructed under the supervision of Mason D. Pratt, M. Am. Soc. C. E. This tunnel passed through the property of the Central Iron Works, on parts of which there were large slag dumps. The water flowing into the tunnel was so sulphurous from the sulphur in the slag, that it greatly impeded the progress of the work and was most unwholesome for the workmen. Cast-iron pipe in such a location would undoubtedly undergo rapid deterioration.



SECTION AND VIEW OF CORRODED PIPE FROM ELIZABETH, N.J.

ACTION OF FERROXYL REAGENT ON SAME PORTION OF PIPE

FIG. 21.

Where low lands are filled with cinders, ordinarily, no trouble need be feared, but when subjected to periodical wetting and aeration, some precautions are desirable.

Acid mine waters are also very corrosive, and should be guarded against, when pipe is subject to their action. In the mines themselves this would generally be economically impracticable, but, in municipal works in the vicinity, where the pipe may be attacked, preventive means should be resorted to.

PREVENTION OF EXTERNAL CORROSION.

A recipe to prevent rusting of iron, given in an old work called "The Laboratory", published in London, in 1790, is as follows:

"Fry a middling eel in an iron pan, and when brown and thoroughly fryed, express its oil, and put into a phial, to settle and become clear in the sun. Iron work, anointed with this oil, will never rust, although it lay in a damp place."

The actions of iron under the complex conditions obtaining in Nature are so various, and the observed results so often discordant, that one would fain fall back on some such recipe as this as a cure-all.

Bearing in mind the silicious skin of a cast-iron pipe and its protective effect, and also noting that chromium alloys of iron are resistant to acid attack, it occurs to the writer that some chromium compound such as chromite mixed with the facing sand of the moulds might produce in castings a protective chromium skin.

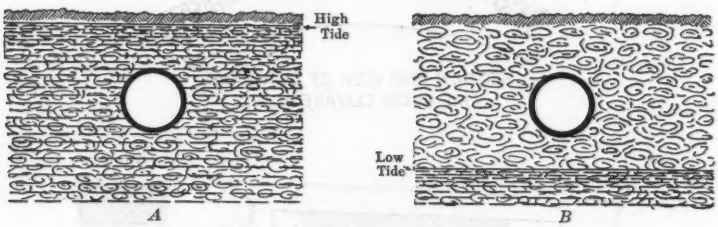


DIAGRAM SHOWING AERATION EFFECTED IN A TIDAL MARSH
A—HIGH TIDE B—LOW TIDE

FIG. 22.

It is a subject wherein an apparently well-reasoned argument may be completely upset. For example, as zinc in water, either fresh or salt, protects iron and is itself attacked, so if iron and copper are immersed in a liquid capable of acting on each, and are connected by a conductor, a current will pass from the copper to the iron, and the iron will be dissolved. And yet, after a series of exhaustive experiments, Mr. Richard H. Gaines* states:

"The legitimate conclusion from the foregoing experiments is that rather less than more galvanic action arises from contact between steel and the other metals of the series than from contact between steel and steel. In actual practice it has been found that if a steel nut is put on a steel bolt and subjected to corrosive conditions, in a comparatively short time the steel nut will have to be hammered or

* *Engineering Record*, Vol. LXIX, p. 104.

split off, if gotten off at all, on account of rusting together from local corrosive action; whereas a brass or bronze nut on a steel bolt under similar condition can be readily screwed off."

Mr. H. J. Force* is authority for the statement that cast iron used in pumping machinery by the Delaware, Lackawanna and Western Railroad Company, was badly corroded by acid mine waters. As bronze bushings, valves, and impeller blades were used, it was thought that the action was due to galvanic action from contact of the two metals in the acid water. On substituting cast iron for the bronze valves and bushings, so that the parts in contact would be of the same metal, the corrosion increased, and the cast iron deteriorated even more rapidly than when in contact with bronze.

As far back as the days of the elder Pliny, who was killed in the same eruption of Vesuvius that destroyed Pompeii in A. D. 79, this effect of iron on iron was noted. Pliny states that Alexander the Great, to repair a bridge over the Euphrates, "sometime bound and strengthened the bridge over the river there; the links whereof, as many as have been repaired and made new since, do gather rust, whereas the rest of the first making be all free therefrom." Modern experience has shown that if the old and new iron are of absolutely similar metal, one will be as durable as the other; but that if they are of a slightly different composition, it is not necessarily either the inferior or the newest metal which suffers most, that depending on the electrical phenomena resulting.

It is unsafe to reason too positively from results obtained with liquid electrolytes as to what will happen to iron buried in soils. Moreover, time is such an important factor that conclusions drawn from accelerated tests may be most misleading. At the beginning of this paper, reference was made to the conclusions of Messrs. McCollum and Logan.†

"The relatively high rate of self-corrosion of cast iron as compared to the other kinds of iron tested is contrary to the generally accepted idea that cast iron is more resistant to self-corrosion than wrought iron. It is not improbable that this impression in regard to the superiority of cast iron has grown out of the fact that cast iron structures are usually made relatively heavy and they also tend to corrode more uniformly than wrought iron or steel, both of which factors would tend greatly to increase the life of the former."

* *Engineering Record*, Vol. LXIX, p. 104.

† "Electrolytic Corrosion of Iron in Soils," p. 32.

The writer cannot subscribe to this view. The ordinarily accepted idea of the superior durability of cast iron arises, in his opinion, from the fact that cast iron, generally speaking, is more durable than other commercial forms of wrought iron and steel.

One explanation of this discrepancy may be furnished by the experiments of K. Arndt,* who determined the extent of corrosion in iron and steel by the oxygen absorbed by the metal in rusting. Fig. 23 shows the result of these experiments.

The cast iron at first corrodes as rapidly as the mild steel, and more than twice as fast as the weldless tube. At the end of 43 days, however, the mild steel had undergone twice as much oxidation as the cast iron, and the weldless tube four times as much.

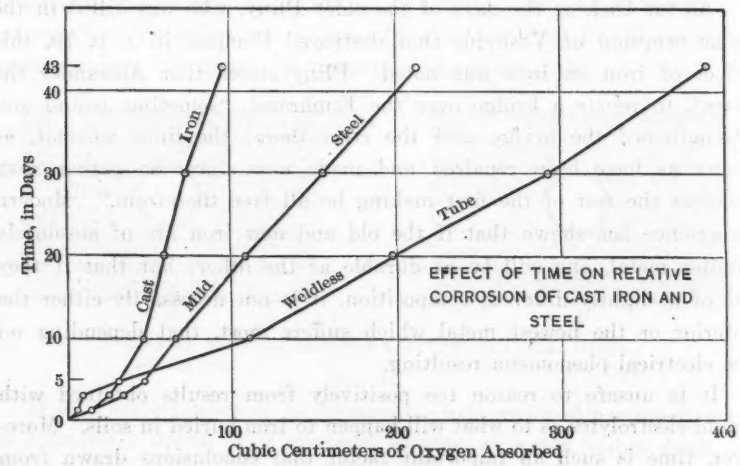


FIG. 23.

It would seem, therefore, that even in the corrosive zones, in which deterioration of the cast-iron pipe has been greatest, the remedy is not to be sought by the substitution of wrought iron or steel, but rather by the improvement, if possible, of the skin resistance of the metal. Where, for any reason, such as with flanged pipe, bolts and nuts are necessary, bronze need not be feared on account of its supposed galvanic effect on the iron.

A second method of protecting the pipe, based on the well-known rust-inhibiting properties of the chromates and certain other substances, whereby the iron is rendered "passive", does not seem prac-

* *Chemiker Zeitung*, 1910, Vol. XXXIV, p 425.

ticable, in that a sufficient quantity of such material to protect the iron indefinitely could scarcely be added to the soil surrounding the buried pipe at a reasonable cost. Therefore, this interesting phase of the subject has not been taken up in this paper. A temporary passivity would be of no practical importance.

The use of lime in the trenches, so as to encase the pipe in an inhibiting alkali, ought to be of service, and is worth trying. Encasing in concrete would be expensive, but would protect the pipe, if properly done.*

Another method, suggested at Perth Amboy, is to leave the pipe in open trench. This might help materially, as the alternate wetting and drying would probably be less destructive than the constant exposure to moist air, salt, and carbonic acid in the swamp or meadow soil. At Atlantic City this principle has been carried one step farther, and the new 48-in., cast-iron, water main across the salt marsh has been raised above the surface of the meadow, so as to be entirely in the air.

As will be seen by Fig. 24, it is supported on reinforced concrete bolsters, 5 ft. 2 in. long, 20 in. thick, and about 5 ft. 6 in. high, so that the bottom of the pipe is about 1 ft. above the meadow. Each bolster, in turn, is carried on two piles of an average length of 37 ft., driven by a 2 000-lb. hammer, the final penetration required being not more than 1 in. under five strokes of the hammer falling 14 ft. The bolsters are 6 ft. from center to center. Between the pipe and the bolster there is a piece of hard vulcanized fiber, $\frac{3}{8}$ in. thick. On the pipe laid in summer, expansion was provided for by allowing a minimum space of $\frac{1}{4}$ in. between the shoulder and spigot end of each pipe. In addition, a sleeve is inserted once in each 2 000 lin. ft. of pipe. The entire design has been carefully studied and worked out by Mr. Lincoln Van Gilder,† Chief Engineer, and T. Chalkley Hatton, M. Am. Soc. C. E., Consulting Engineer of the Water Department. Several thousand feet of the main are already completed. It would seem to solve the problem, either in a warm climate or where the main is large enough to be exposed without danger of freezing; and where it is permissible to have the pipe above the surface.

* For use of cement as a wash, see Franklin Riffle, M. Am. Soc. C. E., Discussion on "The Preservation of Materials of Construction," *Transactions*, Am. Soc. C. E., Vol. L, p. 313 *et seq.* See also Discussion on "Relative Permanency of Steel and Masonry Construction," *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 74, for many interesting data.

† Who furnished this information to the writer. See also *Engineering News*, Vol. LXX, p. 1046.

At Elizabeth an analogous, though less elaborate, expedient has been adopted. A 6-in. pipe could not be raised above the meadow and left exposed without danger of freezing, so it was laid on the surface, covered with a mound of meadow soil or mud some 6 in. thick, and the mound was then sodded with fox grass. This was done in the spring, so that the sod would make vigorous growth and protect the embankment. Although this gives great promise of obviating the trouble, it has not been in place long enough to predict its success with absolute certainty.

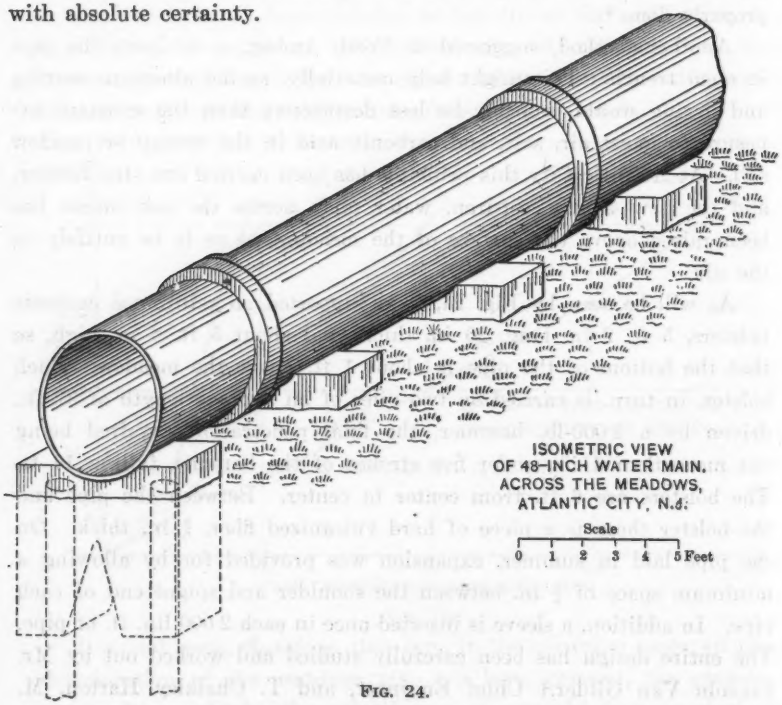


FIG. 24.

Very frequently, conditions are met which prohibit such a solution of the problem. At Syracuse, for example, where the mains are laid in the streets, such a mode of construction is not to be thought of.

Bearing in mind the protective effect of zinc, as shown in Fig. 11, it might at times be desirable to galvanize the cast iron in such locations, instead of giving it the ordinary bituminous coating. The United States Government has utilized galvanizing in some instances for the protection of gratings, covers, and similar castings, but as far as

the writer is aware, it has never been applied to cast-iron pipe. Double galvanizing, as it is called, should be used in such cases. This is a trade term, which does not mean that the iron has been dipped twice, but merely that it carries a much heavier coating than the ordinary galvanized metal. The heavy coating is not suitable for articles which may be subjected to bending, as it will crack off, but this objection does not apply to a rigid article like cast-iron pipe.

Conclusions.

First.—Under ordinary conditions of soil, cast-iron pipe has a probable life of from one to three centuries, as far as external corrosion is concerned.

Second.—Under certain soil conditions, such as salt marshes or saline soils, cast-iron pipe may be rendered useless in from 7 to 20 years.

Third.—At times, cinder and slag fills may exert a strongly deleterious influence. Acid mine waters are also destructive.

Fourth.—Substituting wrought-iron or steel pipe for cast iron is ineffectual. Cast iron will outlast the others.

Fifth.—Remedies fall under four heads:

A.—Increasing the skin resistance of cast iron;

B.—Utilizing the protective influence of alkalis by surrounding the pipe with lime or cement, where such is practicable;

C.—Exclusion of acids, salt, or air;

D.—Galvanizing the cast-iron pipe, thus protecting it at the expense of the zinc.

Sixth.—So many opposing factors are at work simultaneously, that great caution is necessary in reasoning from effect to cause and in suggesting remedies based on observed fact. It is the unexpected that frequently happens.

Acknowledgments.—The writer is under great obligations to the persons who have aided him with information, and to the published articles bearing on the subject. The references in the text will indicate to whom credit is due for this assistance, and to these gentlemen he wishes to express his thanks.

Doubtless much that is pertinent to the subject has not been mentioned, but it is hoped that any such facts may be brought out in the discussion.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE STEREOSCOPIC METHOD OF SURVEYING, AND A FIRST TRIAL OF ITS APPLICATION TO A RAILWAY SURVEY IN CHINA.*

By Georg A. G. Müller, Esq.†

Synopsis.

In order to demonstrate the advantages of the stereo-photographic method of surveying, the writer made a survey for a part of the proposed Hankow-Ichang Railway, through a difficult piece of country in the very rough and tortuous valley of the Yangtze Kiang River, in China. The instruments and methods used are described and illustrated in detail by their application to this survey.

The photo-theodolite is a combination of a metallic box camera and a theodolite; the optical bases are measured with a rod, mounted on a tripod; the three co-ordinates of each point on the negatives (or diapositives) are measured on the stereo-comparator, which, perhaps, is as important an instrument as the photo-theodolite in stereo-photographic work; the points are plotted on a special drawing device consisting of a glass plate to which steel rulers are pivoted and clamped. The other apparatus consists of double plate-holders, photographic plates, etc., etc.

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

† Regierungs Baumeister; Dipl. Ing. (Berlin); Professor of Civil Engineering to the Government University, Peking, China.

The general plan of the survey included a preliminary reconnaissance of the territory, a triangulation network with a base line in the open part of the valley, and a line of levels in the bottom of the valley establishing bench-marks from which the elevations of the photo-theodolite could be determined or checked. The photographic stations or standpoints were then selected, and from these the photographs were taken, either at the time or whenever the weather and the position of the sun were favorable.

On account of the newness and possible importance of stereophotographic surveying, the theory is illustrated and described in considerable detail, and also the manner in which the points on the photographs are measured and plotted.

It is claimed for this method that it is simple, about seven times as rapid as a tacheometrical survey, is not expensive, and is especially suitable for work in rough and difficult territory.

Although the paper relates only to the application of this method to a preliminary railway survey, it may be used for other purposes, for instance, stereographic pictures, taken from a fixed base line, will serve to record the progress of work, as they will enable one to check quantities of earthwork, masonry masses, dimensions, the agreement of structure with design, etc., and may be examined at headquarters at any time.

While making a topographic survey in Hanyang-Wuchang during the summer of 1913, with his students, the writer had an opportunity to demonstrate the stereo-photographic method of surveying to Councillor Linow, Chief Engineer of the Hankow-Ichang Railway. The particular advantages and the results obtained recently in Europe by this method convinced the Chief Engineer that it might be advantageous to make a trial of it in China, and the writer was commissioned to make a preliminary photographic survey of the most difficult part of the proposed line during the summer vacation.

The party consisted of the writer as Chief Engineer, and later two German engineers, six of his former students, now graduated, ten Hupeh students, graduated in Japan, and a few draftsmen.

The preliminary line, proposed after the first reconnaissance, runs for about 20 li* (7 miles) nearly parallel to the Yangtze Kiang,

* 1 li = about ⅓ mile.

thence for 20 to 30 li (7 to 10 miles) through hilly, rolling country, and, finally, after crossing a tributary of the Yangtze, reaches the district under discussion. The valley is extremely rough and tortuous, and, with its steep rocky bluffs and wooded slopes, presents great difficulties to the locating engineer, so that the work served as a valuable test of the possibilities of this method of surveying.

This valley, measuring 10 to 12 li (3.6 to 4.3 miles) on the line and about 17 li (6.1 miles) along the river, represents the *pièce de résistance* of the Hankow-Ichang Line. Probably it will be the most expensive part, on account of the many structures required. For this reason the survey was first commenced on the scale of 1:2 500; the rolling country was to be surveyed later.

General Plan of the Work.

The line of the reconnaissance was marked by flags on high bamboo poles every 400 to 600 m. (1 300 to 2 000 ft.). Then a triangulation net was made over the whole district to be surveyed, starting from a favorably located base line in the broad open part of the valley. It was endeavored to include in this triangulation as many points on the line as possible, and, in order to avoid confusion, such points were indicated by poles with flags at their tops; the poles at the triangulation points also bore flags in the middle. Each flag carried a wooden target at a height of about 1½ m. from the ground. Altogether, about 30 points were established.

A base line of about 214 m. (700 ft.) was carefully measured with 5-m. wooden measuring rods which had been standardized in Peking. From each point the direction of all visible points was determined within 10″ with four readings. At the same time, most of the elevation angles were taken.

The triangulation served a double purpose: First, it made possible the orientation of the standpoints of the photo-theodolite. Secondly, its points (the elevations of which were determined trigonometrically), formed an important control for the photographic plates.

As explained later, this check served, even when the bamboo poles did not appear on the plates. The targets answered as base points for the stations, as the lower parts of the poles were frequently concealed by grass or otherwise, the average height of the theodolite above the ground being 1½ m.

The "distances" of the triangulation (400 to 1 200 m.) (about 1 300 to 4 000 ft.) were calculated and plotted, and precise levels were run along the bottom of the valley. From these the elevations of the photo-theodolite were determined or checked.

The selection of the photographic standpoints was made, the directions and base lines for the stereographic exposures were taken, base lines were measured and oriented with reference to the triangulation, and occasionally a few elevation angles were measured by the photo-theodolite. The points for the camera were marked by pegs, and the photographs were taken, if the weather and position of the sun permitted, or postponed until a more favorable time. The plates were developed and diapositives and prints were made at headquarters.

The next stage of work was the taking of measurements from the plates in the stereo-comparator. This is a most important instrument and on it the progress of the survey depended. It was kept continuously at work during daylight, by shifts of operators, throughout the whole survey.

The determined co-ordinates from each pair of plates were plotted on tracing paper pasted on especially arranged glass plates, and the orienting directions were laid out. Then these sheets (one for each pair of photographic plates, Figs. 18 and 19) were laid on the triangulation map and oriented so that the orienting lines would pass through the triangulation points; the other points were then transferred by pricking, and the elevations indicated.

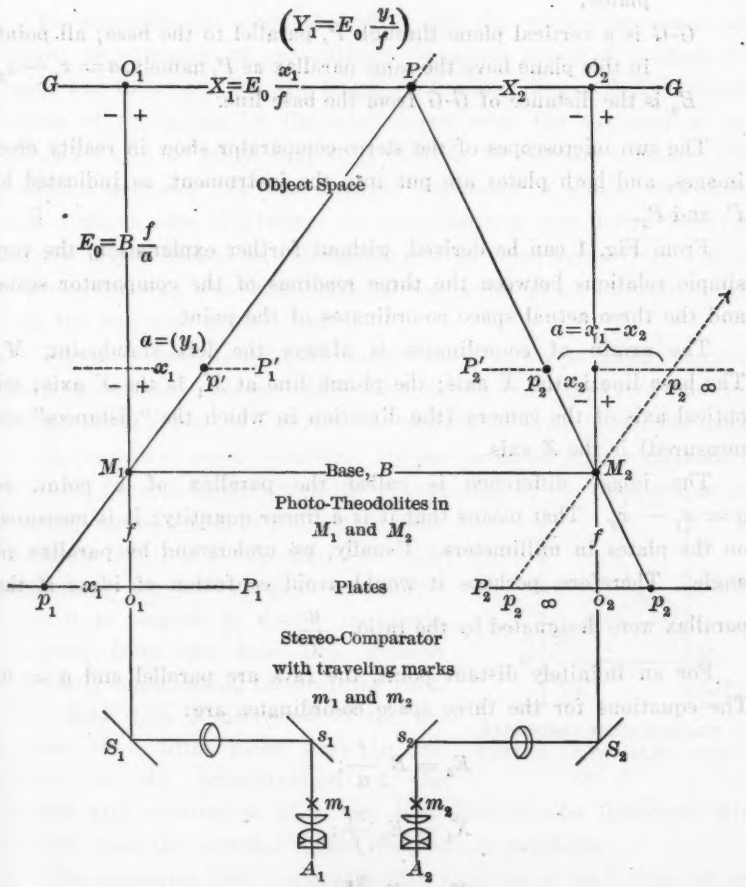
Having transferred several consecutive "point sheets," the drawing in of contour lines was commenced. Finally, the map was taken into the field for comparison with the actual topography, and objects which had been concealed from the camera, or had otherwise been missed, were filled in by free hand or by stadia, according to their size and importance. Also, on this last visit to the field, the topographic signs (buildings, culture, etc.), were indicated on the map.

Then the paper location was made on the original map or on tracings from it.

Theory of the Stereo-Photogram.

The theory of a stereo-photo-grammetrical exposure, in which both plates are in the same vertical plane, is illustrated by Fig. 1, in which M_1 and M_2 denote standpoints of the camera, that is, the center

of the objective. The difference of a few centimeters between center of objective and center of instrument can be neglected.



THEORY OF A NORMAL STEREO-PHOTO-GRAMMETRICAL EXPOSURE

FIG. 1.

M_1-M_2 (= B) is the horizontal projection of the base line;
M_1-O_1 and M_2-O_2 are the optical axes, and are parallel to one another and perpendicular to the plates and to the base line;
f is the focus of the objective;

P is any point to be determined;

p_1 and p_2 are the image points of P on P_1 and P_2, the negative plates;

G-G is a vertical plane through P, parallel to the base; all points in this plane have the same parallax as P, namely, $a = x_1 - x_2$;

E_0 is the distance of G-G from the base line.

The two microscopes of the stereo-comparator show in reality erect images, and both plates are put into the instrument, as indicated by P_1 and P_2.

From Fig. 1 can be derived, without further explanation, the very simple relations between the three readings of the comparator scales and the three actual space co-ordinates of the point.

The origin of co-ordinates is always the left standpoint, M_1. The base line is the X axis; the plumb line at M_1 is the Y axis; the optical axis of the camera (the direction in which the "distances" are measured) is the Z axis.

The image difference is called the parallax of a point, as $a = x_1 - x_2$. That means that it is a linear quantity; it is measured on the plates in millimeters. Usually, we understand by parallax an angle. Therefore, perhaps it would avoid confusion of ideas if the parallax were designated by the ratio, $\frac{a}{f}$.

For an infinitely distant point, the rays are parallel and $a = 0$. The equations for the three space co-ordinates are:

$$E_0 = B \frac{f}{a};$$

$$X_1 = E_0 \frac{x_1}{f};$$

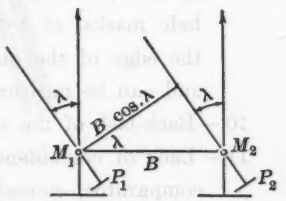
$$Y_1 = E_0 \frac{y_1}{f}.$$

It is essential in normal photo-grammetry that the two plates be in the same vertical plane. However, points can be determined from two plates not in the same plane, but, it might be mentioned, parenthetically, that in any case, with the instruments herein mentioned, a base line is used which is only from one-twentieth to one-tenth of the length of that required by the old photographic method.

The two plates are used in conjunction to determine the distance of a point, and the left-hand plate alone serves to determine its direction and elevation.

In principle, the photo-theodolite and the stereo-comparator are simply an extension of our ordinary binocular power of judging distance and direction; but the length of the base line and the magnification of the image by the microscopes raise the efficiency of our natural vision so much that it is possible to measure distance with sufficient accuracy for map-making. Although a single view gives only such an idea of distance as our experience can derive from the relative apparent size of objects, the second view, taken from a point removed laterally somewhat from the first, gives us an accurate insight into the structure of the view.

The most used, because the most simple, is the so-called "normal" stereo-photogram. Nevertheless, the scientific collaborator of the Zeiss Werkstätten, Jena, Dr. Pulfrich, has thoroughly investigated the theory of the curves of equal parallax. He has invented simple methods of plotting for exposures, where the optical axes of the photographs are equally turned either to the left or right (usually 30°), Fig. 2; therefore such exposures are of increased usefulness, and it is possible to double the angle covered from one base line without seriously complicating the drafting work. The field work is decreased at the expense of a little more work in the office, as the determination of the length and orientation of a new base line can be dispensed with. In this case, the curves of equal parallax are parabolas.



EXPOSURES WITH EQUALLY TURNED, OR PARALLEL, AXES

FIG. 2.

The exposures with horizontal axes inclined to each other at any angle, that is, convergent, if both axes are directed to one point in the foreground, is very similar to the old method of plane-table photogrammetry. This method, however, is seldom used, owing to the difficulties of plotting, which are not yet overcome. The curves of equal parallax are ellipses. The construction of one point requires too many auxiliary lines. It would go too far afield to enter here into a discussion of the accuracy of stereoscopic vision with our

two eyes, on the ground of recent physiological investigations. Also, the writer has not the space here to derive a complete theory as to the errors in measuring with the stereo-comparator; it may suffice to mention, among other various sources of error, the following:

1.—Inaccurate determination of the base line in the field;
2.—Inaccurate determination of the focus of the objective by the makers (all views are made with a fixed-focus camera);
3.—The lens not perfectly free from optical distortion;
4.—Too coarse-grained emulsion on the plates (which must be especially prepared);
5.—Not exactly vertical position of the plates during exposure;
6.—A slight inclination of the plates in relation to each other;
7.—A slight inclination of the axes in relation to the base line, though they are still parallel to one another;
8.—Not perfectly plane plates (especially with long-focus lenses);
9.—Expansion and contraction of the plates by change of temperature; or a creeping of the gelatine film, for a like reason, or by moisture. (A check against the latter is provided by two hole marks, at a fixed distance apart, which are printed on the edge of the plate automatically at the time of exposure, and can be measured later under the stereo-comparator.)
10.—Back-lash of the screws of the comparator;
11.—Lack of coincidence of the planes of the two pictures in the comparator, caused, for instance, by difference in thickness of the glass plates;
12.—One or both of the plates not resting exactly in a horizontal position in the comparator.

Obviously, it is necessary for the surveyor to understand and take into account how much each of these sources may affect the accuracy of the result. It is necessary and possible to be able by checks to determine the sources of errors and make proper adjustments accordingly.

Instruments.

The stereoscopic method is very sensitive, and the instruments used must be of very great precision. The only manufactory of these instruments is the well-known Zeiss Werkstätten, Jena. The instruments

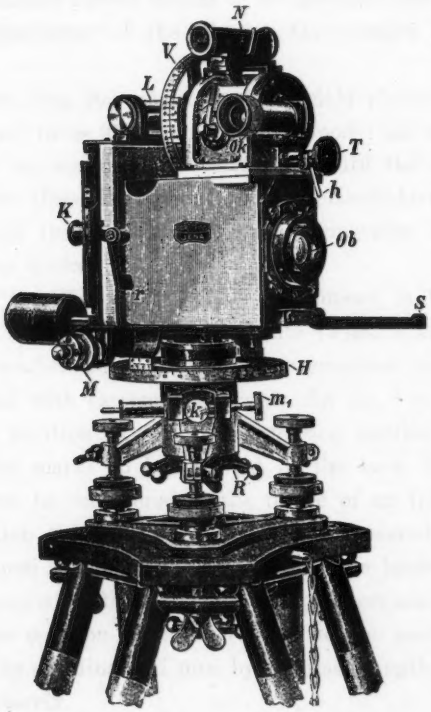


FIG. 3.—FIELD PHOTO-THEODOLITE.

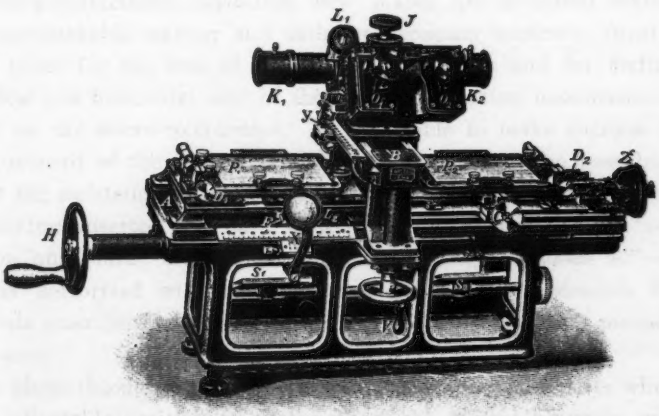
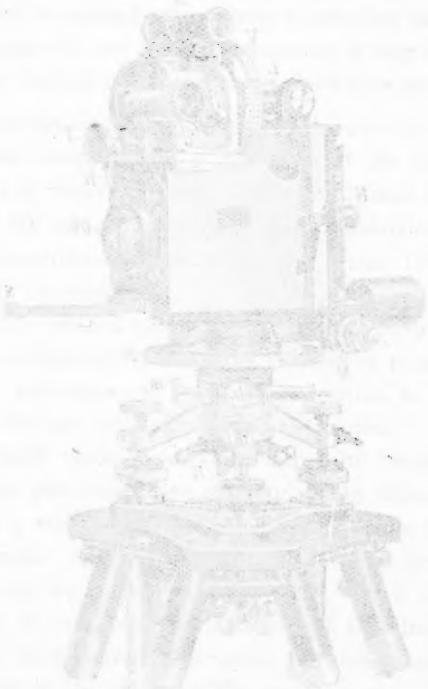


FIG. 4.—STEREO-COMPARATOR BY DR. PULFRICH.



used in this particular survey belong to the geodetic outfit of the Civil Engineering Department of the Chinese Government University at Peking.

The equipment, Fig. 20, consisted of one field photo-theodolite, for 9 by 12-cm. plates, focus 98.54 mm.; three tripods; one measuring rod for optical base measurement in connection with the directing telescope of the photo-theodolite; twenty double plate-holders; one stereoscopic range-finder (power $4x$); one stereo-comparator, Model D; and a special drawing device.

The Field Photo-Theodolite.—This instrument is a combination of a metallic box camera with a theodolite (Figs. 3 and 5), so that the particular conditions of a "normal" stereogram can be fulfilled expeditiously and with extreme accuracy. An error of from 1′ to 2′ in the relative position of the plates to one another is much too great. Then the marks to be adopted in the back frame in front of the plate must be considered as the image of an infinitely distant point, from which the parallaxes are to be measured, for instance, exactly to 0.01 mm., if known fixed points in the landscape are to be dispensed with and corrections avoided. The highest admissible angular deviations in the position of the two plates to one another, therefore, are determined by dividing 0.01 mm. by the focal length, $f = 180$ mm., or 10″, approximately.

Instead of the vertical hairs previously used in the camera frame for photo-grammetrical exposures, new marks are produced which, in an unmistakable manner and with the necessary accuracy, furnish a fixed point for the zero of the values of parallax and for finding the optical and horizontal axes of the plate when taking measurements from it on the stereo-comparator. It is possible to make changes in the adjustment of the camera to the horizontal and to the base line, without the assistance of the marks in the camera frame, at any time after having inserted the dark slide. The adjustment of the two plates in one plane—to be done with an accuracy of about 10″—is not only performed with ease and precision, but is accessible for immediate examination and rectification, right up to the very moment of exposure.

The photo-theodolite proper consists of three principal parts which are in adjustable position to one another: the central bearings, with

the horizontal circle, insertable in the tripod, the camera firmly fixed to the axis of rotation, and the pointing telescope with the vertical circle.

The horizontal and vertical circles are divided into half degrees, the vernier permitting readings of 1′ and estimations of ½′. The horizontal circle is adjustable—coarse by hand and fine by micrometer screw—for the purpose of setting on a certain zero point.

The vertical axis of rotation can be firmly clamped and fine adjustment effected with a micrometer screw, which is also designed for the measurement of the base.

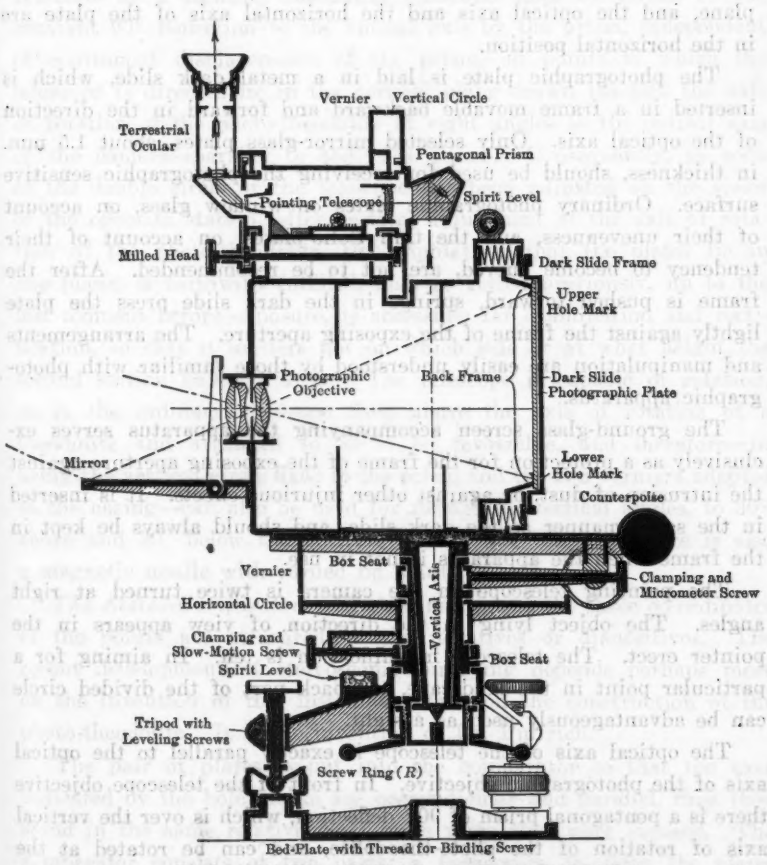
The camera casing is of light metal, cast in one piece, into which the objective is fixed in front; the back frame for the sensitive plate is behind. The optical axis is at right angles to the plane of the back frame at a certain point mentioned later.

The ortho-protar objective in the photo-theodolite is particularly suitable for photo-grammetrical and stereo-photo-grammetrical exposures, on account of its simple structure and consequently its ability to resist external physical influences, its fine definition from the center to the margin of the plate, and its well-nigh perfect orthoscopy —the deviations are demonstrably smaller than 0.01 to 0.02 mm. The horizontal image angle is more than 60°, so that in wide-angle views only six exposures—in which the margins somewhat overlap one another—are necessary for the whole circuit. The immovable objective is below the middle of the exposing aperture of the back frame. In cases where only one photo-theodolite is used, the objective is provided with a fixed stop of $\frac{f}{40}$. The focal length of the objective is most carefully determined in the works, up to 0.1 mm. for each instrument, and stated on delivery of the apparatus.

The back frame is ground perfectly plane, and adjusted so that all around the photographic plate a few millimeters are blocked out. It is exactly parallel to the axis of rotation of the photo-theodolite.

The new marks in the back frame are to be seen on the photographs, Figs. 18 and 19. They consist of two circular holes, with conical embrasures, turned toward the objective, about 0.15 mm. in diameter, one of which is in the middle of the upper margin of the frame, the other in the middle of the lower margin. The line joining the two holes is directed parallel to the axis of rotation of the photo-theodolite, and cuts the optical axis in one point.

The hole marks have this advantage, over the knife-edges generally used, that their centers undergo no displacement whatever through differences in illumination, and can be easily rediscovered under the microscope of the stereo-comparator with the necessary accuracy.



DIAGRAMMATIC VERTICAL SECTION THROUGH THE PHOTO-THEODOLITE

(Pointing telescope is drawn with the ocular in the vertical position)

FIG. 5.

Because of the adjustment mentioned, the vertical axis of rotation of the photo-theodolite, the optical axis of the camera, and the horizontal of the plate—the last is determined by the reading on the y-scale and by the slide rests of the stereo-comparator lying at right

angles to one another—form a fixed rectangular co-ordinate system. There is required then only the exact vertical setting of the axis of rotation, which is done with the set-screws on the tripod and the 10-in. spirit level on the camera, and then the plate lies in the vertical plane, and the optical axis and the horizontal axis of the plate are in the horizontal position.

The photographic plate is laid in a metal dark slide, which is inserted in a frame movable backward and forward in the direction of the optical axis. Only selected mirror-glass plates, about 1.5 mm. in thickness, should be used for receiving the photographic sensitive surface. Ordinary photographic plates on window glass, on account of their unevenness, and the thin Solio-plates, on account of their tendency to become curved, are not to be recommended. After the frame is pushed forward, springs in the dark slide press the plate lightly against the frame of the exposing aperture. The arrangements and manipulation are easily understood by those familiar with photographic apparatus.

The ground-glass screen accompanying the apparatus serves exclusively as a protection for the frame of the exposing aperture against the intrusion of dust, or against other injurious effects. It is inserted in the same manner as the dark slide, and should always be kept in the frame when the apparatus is not in use.

The pointing telescope on the camera is twice turned at right angles. The object lying in the direction of view appears in the pointer erect. The telescope magnification is ten. In aiming for a particular point in the landscape, the back part of the divided circle can be advantageously used as a sight.

The optical axis of the telescope is exactly parallel to the optical axis of the photographic objective. In front of the telescope objective there is a pentagonal prism of 90° deflection, which is over the vertical axis of rotation of the photo-theodolite, and can be rotated at the same time as the twice-turned terrestrial ocular about the optical axis of the telescope. The telescope objective and the cross-lines take no part in the rotation. These stand in connection with the pentagonal prism and the ocular only through the roof of the camera chamber, and form a perfectly closed-in pointer telescope, secured against lateral displacements by screws and pins and against external influences by the hollow body carrying the prism and ocular. With

the assistance of this pointer, the direction of the optical axis of the photographic objective can at any moment be fixed.

In consequence of these arrangements, the pointer is simultaneously available as a sighting theodolite telescope, and, by virtue of the constant 90° deflection to the optical axis by the prism, independent of contingent displacements of the prism, all points to which the telescope is directed lie in the vertical plane drawn through the axis of rotation of the photo-theodolite at right angles to the optical axis of the camera—parallel to the back frame. Consequently, as soon as the double circle of the telescope has been adjusted on the visier of the opposite station, after vertical adjustment of the axis of rotation of the photo-theodolite, the stipulation, that the plates lie in one plane, is forthwith fulfilled, and, as stated previously, up to the last moment before exposure, is accessible for examination and rectification, so that it matters not on which side or at what height the second station happens to be. The telescope is capable of rotation, as in the ordinary telescope fixed above the axis of rotation of a theodolite and arranged so as to be revolvable, and therefore—in using the vertical circle fixed to the prism and the two verniers adapted to the casing—can also be used for measuring vertical angles, to 30° above and 30° below the horizon. Above the telescope there is also a magnetic needle with turned up point.

The Stereo-Comparator.—By this instrument the three co-ordinates of the points are measured on the negatives or diapositives. The recent development of stereoscopic surveying depends perhaps more on the invention of this instrument than on the construction of the photo-theodolite. It is the invention of Dr. Pulfrich.

The pair of plates is put into the comparator so that the axes indicated by the hole marks are perpendicular and parallel, thus they stand in the same relative position as when they were exposed. The comparator consists of two parts; a framework to carry the plates and a binocular microscope for viewing them.

In detail, the construction of this instrument, Fig. 4, is as follows: A heavy bed, A, has a track on which the plate-carrying frame slides in a lateral direction. Surmounting this is a second track, B, at right angles to the first, on which the microscope carriage slides. These motions serve for measuring the two co-ordinates which are parallel to the plates. The remaining co-ordinate (E_0, Fig. 1) is

determined by measuring the parallax by an independent lateral motion of the right-hand plate.

In placing the plates, P_1 and P_2, in the frame, they are adjusted so that the lines connecting the hole marks at the top and bottom of the plate are parallel to the track, B. For this purpose, the plate-holders are mounted on the plate carriage so that they can be turned through a small angle by slow-motion screws, D_1 and D_2. In order to compensate for the difference in elevation of the two standpoints from which the plates were taken, the right-hand plate is adjustable vertically by the screw, C.

Lateral deviation is measured by the scale, X, vertical deviation by the scale, y, and distance, that is, parallax, by the scale, Z, to 0.001 mm. The parallax measurement must be made with the utmost accuracy, as the other two co-ordinates are derived from it. Its importance is analogous to that of quick and accurate distance measurement in tacheometry.

For viewing the plates there is a Helmholz tele-stereoscope in connection with a binocular microscope (power $6x$ and specific plastic $4x$). In order to be able to measure with the tele-stereoscope in the image plane of both eyepieces in the center of the field, a balloon-shaped measuring mark is fixed. This mark resembles a solid circle standing above a plus sign. When properly adjusted, the stereoscopic effect causes these two images to coincide and appear as a single image floating in space. With the shifting mechanism, the observer is able to place this mark on any desired point in the landscape. By turning H, it appears to move laterally; by turning V, it appears to move vertically; by turning Z, it appears to move to and fro. This artificial mark has the function of the rodman in a tacheometrical survey, with this difference, that it depends solely on the will of the observer and not on the capacity of the rodman; also, it is not influenced by wind or weather, and is not handicapped by roughness and inaccessibility of the territory to be surveyed.

It is important to notice that it is not necessary for the observer to have the power of stereoscopic vision, because, in placing the balloon mark on a certain point in the landscape, he first sets the point in the left ocular, then brings the mark into coincidence in the right ocular, before making a binocular observation, as in the case of those with one defective eye or with one eye especially trained.

Plotting Device.—The device for the plotting of the map (Fig. 6) is a strong mirror-glass plate on which a steel ruler, L_1, is pivoted at a point representing the left standpoint, M_1. There is also a ruler, L_2, which can be clamped in a direction parallel to the optical axis. Both are of thin saw-blade steel.

The Range-Finder.—The stereo-comparator has an advantage over another very useful surveying instrument, the Zeiss range-finder. The latter, invented and perfected by Dr. Pulfrich, was found very useful in checking the side measurements of the triangulation, the check measurements on the more important points in the photographic survey, and also in the preliminary measurement of distances in order

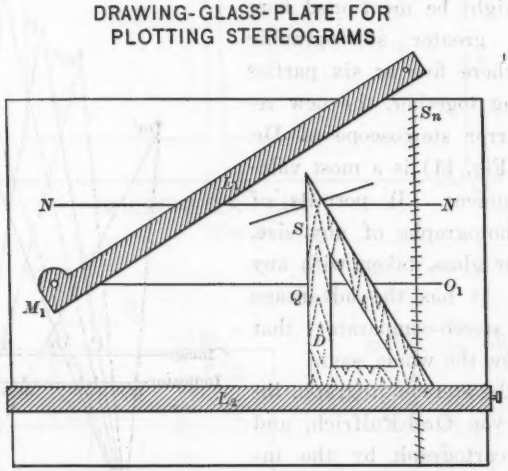


FIG. 6.

to make a proper selection of the base lines for the photographs, etc., taking into consideration the scale of plotting. The observer sees in the range-finder a stadia scale assuming the appearance of a solid zigzag line floating in space (Fig. 7). By this scale he is enabled to determine the distance of any point, "as if counting the poles of a telegraph line." He brings into coincidence with the point in question that mark on the scale which is apparently at the same distance. The range-finder in its usual form has a fixed scale.

The Stereo-Micrometer.—Another quite useful instrument, but of more theoretic-pedagogic interest, is the stereo-micrometer. It serves exceedingly well to explain the principle of the "wandering

or traveling mark" (Figs. 9 and 10) and the stereoscopic method of measuring distances in general. In this micrometer there is a "wandering mark" and pictures in fixed position relative to each other; in the stereo-comparator there are two fixed marks under which the plates are movable. The effect is practically the same.

Accuracy in working with the range-finder depends very much on the capability and practice of the observer. It might be mentioned that the ordinary stereoscope can only be used for small pictures taken with an ordinary camera with objectives separated by the usual interpupilary distance of 65 mm. Also, it might be mentioned here that, for greater stereographic surveys, where five or six parties are working together, the new reflecting-mirror stereoscope of Dr. Pulfrich (Fig. 11) is a most valuable instrument. It permits of viewing photographs of any size, on paper or glass, taken with any base line. It has the advantage over the stereo-comparator that one can view the whole scene.

FIXED MARKS IN THE ZEISS RANGE-FINDER.

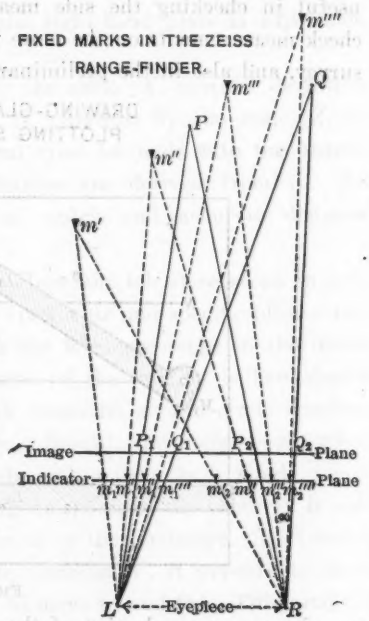


FIG. 7.

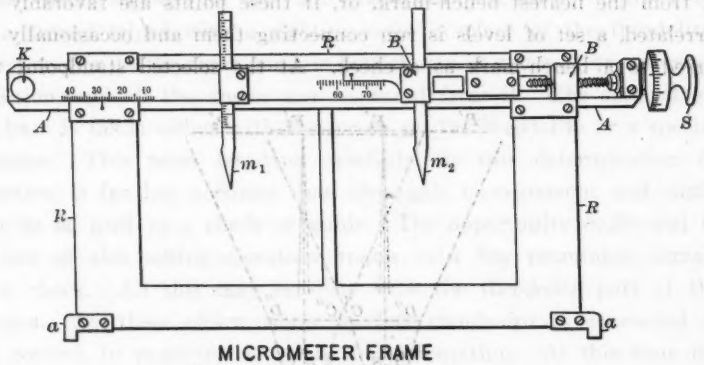
About the stereo-autograph, invented by von Orel-Pulfrich, and the stereo-cartograph by the inventor of the comparator, something may be said later, because they do not belong to the ordinary surveying outfit on account of their high cost.

GENERAL METHODS OF WORK.

It is necessary to distinguish between the work done in the field and that done in the office.

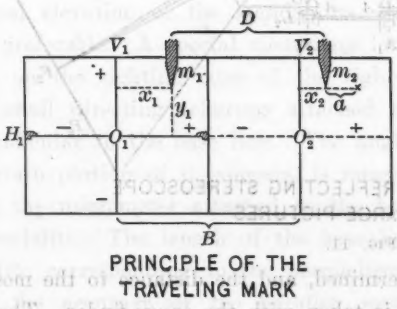
Field Work.—The outdoor work consists in the preparatory survey, that is, in selecting suitable and elevated auxiliary points. It is frequently desirable to run a line of bench-mark levels through the district to be surveyed. These can be referred to when necessary.

The natural characteristic points in the landscape, or the artificial signals to be erected, must be situated so that the resulting intersections are favorable for the triangulation; on the other hand, it is necessary that three, or better more, are visible from each standpoint, for the purpose of orientation and eventually for taking sights to the elevation marks.



MICROMETER FRAME

FIG. 8.



PRINCIPLE OF THE TRAVELING MARK

FIG. 9.

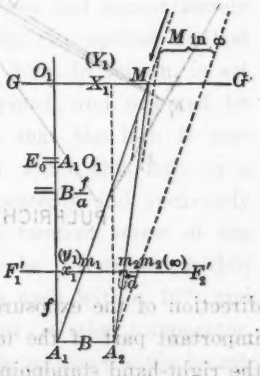
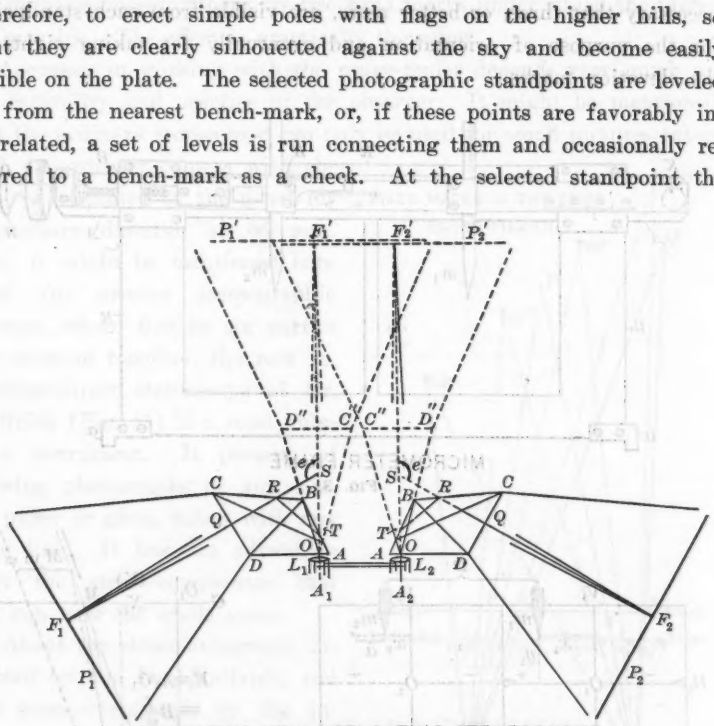


FIG. 10.

It is very desirable, but not always possible, to have such elevation control points on each plate. In selecting the trigonometrical points, the plan of attack with the camera must be considered beforehand. Occasionally, it is better, or even necessary, first to select the photographic standpoints, in order to be sure that they can be oriented in the auxiliary net thrown over the territory by resectioning or other-

wise. It is desirable, of course, to have these points visible on the plate. The erection of characteristic signals, as in first-class triangulation, would be too expensive for smaller distances. It is better, therefore, to erect simple poles with flags on the higher hills, so that they are clearly silhouetted against the sky and become easily visible on the plate. The selected photographic standpoints are leveled in from the nearest bench-mark, or, if these points are favorably interrelated, a set of levels is run connecting them and occasionally referred to a bench-mark as a check. At the selected standpoint the



PULFRICH'S NEW REFLECTING STEREOSCOPE FOR LARGE PICTURES

FIG. 11.

direction of the exposure is determined, and the distance to the most important part of the territory is taken with the range-finder. Then the right-hand standpoint is fixed at the length of the base in a direction perpendicular to the optical axis of the first exposure.

The base line bears a certain ratio to the average distance of the chief objects in the view and to the scale on which the map is to be plotted. Nevertheless, one need not be too strict, as the natural lay of the land usually renders a little more or less distance necessary. It is desirable that all stereograms of any one survey be of uniform accuracy.

It has proved very useful to mark these standpoints carefully. This makes it possible to return to the field and repeat exposures which have not proved entirely successful photographically, or to choose more favorable conditions of weather and light, without the necessity of repeating auxiliary measurements.

The photo-theodolite on the left standpoint gives the direction to all visible signals; on the right standpoint, higher or lower, is set up a second tripod with a sighting cone in place of the theodolite. The direction to the right point is best chosen at the zero or reference direction, and all the angles are measured from it. The azimuth of the base is taken either with a compass on the theodolite or a special compass. This must be done carefully, as this determination of direction is far less accurate than the angle measurement and ought only to be used as a check or guide. The opportunity ought not to be lost of also taking elevation angles to a few prominent signals as a check. All this may be done with the theodolite part of the camera. By these observations the left standpoint is connected to the control, in position, elevation, and orientation. At this time the base line measurement can be made. Direct steel tape measurement is possible with a short length of favorable ground and approximately equal elevation of the standpoints, but usually the optical method is preferable. A special measuring lath, 1 or 3 m. in length, is set up on the sighting cone of the right-hand tripod, and oriented by a small directing telescope attached to it so that the lath is perpendicular to the base line. The angle under which this lath, or a certain portion of it, appears, is measured repeatedly and accurately by the micrometer attached to the horizontal tangent screw of the theodolite. The length of the base line must be determined within 0.1%, corresponding to the parallelism of the axes within 10″, and to the accuracy of the parallax measurement in the comparator. According to the previously mentioned equations, an error in the length of the base enters proportionally and directly into all co-ordinates. The elevation angles to the right standpoint having been taken, the reduction of the base line to the horizontal can be done in the office. It must be expressly mentioned that the tripods should be set firmly. In hard and rocky soil, or in elevated positions exposed to the wind, extreme caution should be taken in this matter. Finally, the height of instrument (averaging 1.30 m.) from peg to center of objective, should

be measured. Having completed these auxiliary measurements, the actual exposure of the plate can be made, if the weather and sun permit.

The numerating device in the camera is set so as to record photographically the proper number of the exposure. Then the plate-holder is inserted, the slide is removed, and, by a special device, the plate is brought to an even contact with the plain brass rim of the back of the camera chamber, and locked in this position. Just before unstopping the lens, a final observation is made with the telescope pointing to the sighting cone at the other end of the base line, to make sure that the plate is in its proper plane. Immediately thereafter, the exposure is made, with all precaution against disturbing the apparatus.

It is advisable to make all the observations carefully, and always in the same sequence. An omission may necessitate remeasuring. After the exposure, the plate is drawn from its seat on the brass frame, the slide is inserted, and the holder is removed from the camera. Additional exposures can then be made at 30° with the first, or, if these are not desired, the camera and sighting cone are interchanged between the two ends of the base line. This is done without disturbing the tripods. On the right-hand standpoint the exposure can be be made without making further measurements, as the camera has been leveled and oriented.

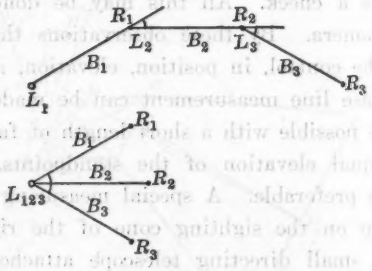


FIG. 12.

In order to make use of some especially favorable elevation for taking a panoramic survey, it is possible to use a series of base lines, either connected in a simple polygon or radiating from a single point (Fig. 12). In the latter case, the left standpoint serves for all base lines, and, of course, very greatly simplifies the field work; this, however, is not always possible on account of the lay of the land, as the selection of even a single base line is sometimes difficult.

A third possibility, recently used quite frequently, is, as already mentioned, making photographs equally inclined (usually at 30°)

to the base line, but still with the optical axes parallel. The writer always works with the horizontal axis, and prefers rather to shift the objective vertically, in order to make full use of the plate, or to bring the important features to the center of the plate, where the drawing is most accurate. It is best to set off the 30° deflection angle from the zero point of the circle, so that any error in the division has no influence on the parallelism of the axes, which must also be assured within 10″. It may be mentioned that, according to the investigation of von Hübel and Pulfrich on stereograms with parallel axes, a small deviation of both axes in the same direction gives a much smaller error than a convergence. Rough sketches, to indicate the territory included by an exposure, are convenient guides in making subsequent exposures during the interval before the development of the first ones. This will prevent the omission of narrow strips, or too much overlapping of consecutive pictures. With this the chief part of the field work is finished.

In surveying railway lines in mountainous districts, one may distinguish two principal cases:

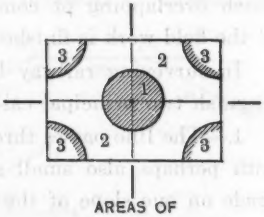
1.—The line passes through a fairly straight and broad main valley, with perhaps also small side valleys, and is on a fairly continuous grade on one slope of the valley. This is the most favorable case for the photographic survey, as the work can be done from the opposite side of the valley, with the different pairs of photographs, taken in approximately the same direction from nearly the same level.

2.—The line passes through a very tortuous valley, or across the topography, so that it frequently crosses the valleys. In this case the photographs must be made in all directions, and frequently from stations where it is difficult or impossible to include all points, especially those in the bottom of the valley, without an expenditure of too many plates which cover only small areas. In such a case it is best to commence the field work with a tacheometric survey of the lower ground, and, with this as a neucleus, to spread the survey by photographic means over the remaining higher territory. A combination of a rapid continuous tacheometric survey of the river with a photographic survey of all the higher and steeper parts is the most economical method. The contour lines are easily and safely developed from the natural guide line—the course of the river—thereby giving ample opportunity to check on the controls. There will be standpoints

near the bends where photographs can be taken looking into and over the valley for certain distances.

Office Work.—First, the triangulation must be calculated and plotted, and the trigonometric levels determined. The net having been plotted, say, on a scale of 1:10 000, copies of it should be issued to each field party, when several are simultaneously at work, in order to avoid confusion. The base lines, elevations of standpoints, orientation angles, etc., are calculated in the office. The standpoints must be denoted clearly and specifically in order to correspond to the numbers registered on the plates by the camera, independently of the plate-holder numbers. Base line No. 7, for instance, has a plate pair with the numbers I-3 and I-4, so that 50 base lines and 100 plates can be denoted by the numerating device.

In the purely photographic part, such as developing, intensifying, or reducing the negatives, making diapositives, prints, enlargements, etc., the photographer has to aim at clearness and accuracy, rather than artistic effect. If the staff is not skilled in photography, it is best to do this work in the field, in order to be able to replace spoiled plates without delay.



AREAS OF DIFFERENT ACCURACY

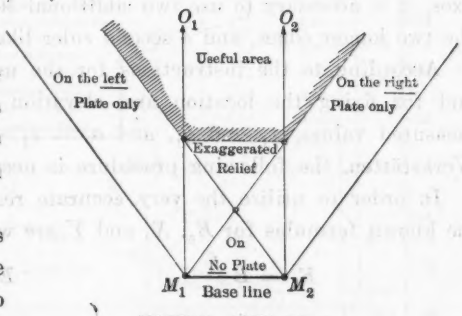
FIG. 13.

The chief work in the office is the measuring of the plates under the stereo-comparator. After a short practice it is possible to measure directly from the negatives. They are more exact, as all reproductions must lose in accuracy and sharpness. Nevertheless, it is advisable to make diapositives, first, in order to insure against the loss or breakage of the negative, and secondly, because sometimes it is possible to obtain good diapositives from weak negatives.

The value of a plate depends on the following considerations: All points which appear on one plate only, though that plate may be a good one, cannot be measured, as they cannot be viewed stereoscopically. Figs. 13 and 14 show the areas of relative accuracy and usefulness on any plate. The points of the very near foreground do not appear on any plate. For instance, those on the right plate and the left border of the left plate are useless. Those of the farther foreground show exaggerated relief, and are not suitable for measuring, as the parallax is too great. Points at very great distances become too in-

accurate, but can eventually be used for the mere topographical completion of the map.

In measuring the plate, the control points are considered first, if they are visible on the plate; and their co-ordinates are measured and compared with those on the triangulation map. If they are not visible, the distance, lateral deviation, and elevation are taken from the map, and the comparator is set to the corresponding scale readings, in order to see where the point falls in the view. Occasionally, it may happen that the image of the point can then be discovered near or coinciding with the balloon mark. At least, it can be observed whether the balloon mark assumes a position on the surface of the ground, or an impossible position in the air, or beneath the surface. Disagreements or errors must be first explained or removed; such precautions are absolutely necessary in order to avoid absurdities. After making sure that the check points agree, the measurements for the other points can be taken.



USEFUL AREA OF A NORMAL STEREO-EXPOSURE

FIG. 14.

In selecting the points needed for drawing the contours, either of two principles may be adopted: All points of equal parallax may be taken, or points of equal y-reading. This simplifies the work of noting and plotting by the elimination of one variable, in each case. The chief objection to such a method is that it is not flexible enough. It is similar to the old cross-profile method. The other principle is to make, in the shelter of the office, a tacheometric survey. This requires the judgment of the experienced topographer, and is a little more troublesome, as all three variables must be considered. However, this method was preferred by the writer for "normal" stereograms. When working in shifts, or in any case to avoid repetition or confusion, the points taken out may be marked on a print in red. The observer with the comparator must consider the scale of the map, and must in imagination travel into the scenery and select fairly evenly dis-

tributed points. Though the parallax can be read to 0.001 mm. and the x- and y-scales to 0.01 mm., it is useless to read to a greater degree of accuracy than can be plotted on the map.

A numbered list of the points, with their co-ordinates, is made and from it the points are plotted.

The plotting device consists of a glass plate, about 1 m. square, with paper pasted on both sides (Fig. 6). On this are drawn the divisions required for the construction of the points (Fig. 15). There is a metallic ruler, L_1, with millimeter divisions on the upper edge and pivoted at M_1 on a conical bearing attached to a metallic inlay in the glass plate. There is also a ruler, L_2, which can be clamped to the plate, and a triangle, D. It is arranged so that the plate can be used on either side. For the plotting of stereograms with inclined axes, it is necessary to use two additional 30° triangles with scales on the two longer edges, and a second ruler like L_2.

According to the instructions for the use of this drawing device, and for fixing the location and elevation of a point, P, from the measured values, x_1 and y_1, and $a = x_1 - x_2$, issued by the Zeiss Werkstätten, the following procedure is necessary:

In order to utilize the very accurate readings of the comparator, the known formulas for E_0, X, and Y are written as follows:

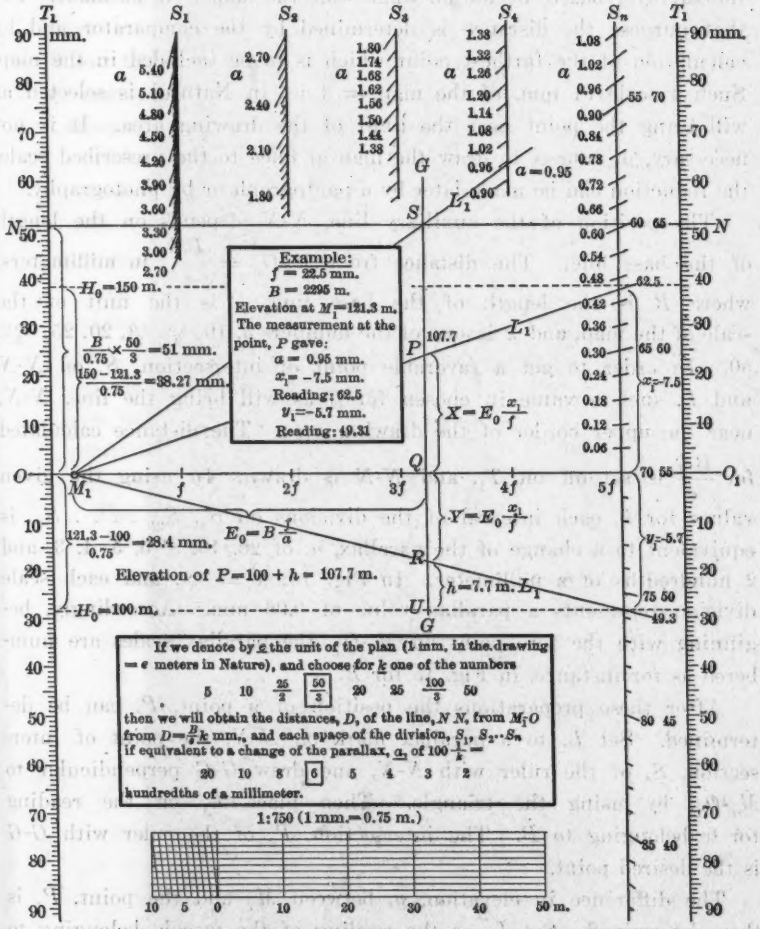
$$E_0 = B\frac{f}{a} \qquad E_0 = k\,B\frac{n\,f}{k\,n\,a}$$

$$X = E_0\frac{x_1}{f} \qquad X = E_0\frac{n\,x_1}{n\,f}$$

$$Y = E_0\frac{y_1}{f} \qquad Y = E_0\frac{n\,y_1}{n\,f}$$

The coefficients, k and n, will be explained later. M_1 always represents the left-hand station, M_1-O_1 the direction of the optical axis. Lay off from M_1 on M_1-O_1 (Fig. 15) the distance, f, the focus, in millimeters, repeated to the edge of the drawing. Through the points thus obtained draw perpendicular lines, S_1 S_2, . . . etc. Divide these S-lines so that the divisions on S_1 = 1 mm., on S_2 = 2 mm., on S_n = n mm., etc. The last line is divided throughout its length, the others only at the upper extremity. T_1 and T_1 are two millimeter scales with zero points on M_1-O_1. Thus each plate is prepared once for all. The actual plotting is done on tracing paper mounted over this. The cone pivot of the ruler, L_1, remains un-

covered. The scale, S_n, is now numbered for x and also for y, but the point on M_1-O_1 is not denoted by zero. The balloon mark of the comparator is set to cover the hole marks on the left topographic



LOCATION AND ELEVATION OF A POINT, P, BY MEANS OF THE MEASURED VALUES, x_1, y_1, $a = x_1 - x_2$

FIG. 15.

plate and then the verniers of the scales, x and y, are adjusted to read whole numbers. These readings are then the respective x- and y-values to be given the middle point of the scale S_n (70 and 55 in

Fig. 15). Starting up and down from this point, the remaining points of S_n are numbered. Both rows of figures are written on the right side of S_n. The figures on the left are to denote the parallax. Now the surveyor has to decide on what scale the map is to be made. For that purpose the distance is determined by the comparator and by calculation of the farthest point which is to be included in the map. Such a scale (1 mm. of the map = 1 m. in Nature) is selected as will bring the point near the limit of the drawing area. It is not necessary, of course, to draw the map at once to the prescribed scale, the reduction can be made later by a pantograph or by photography.

The position of the auxiliary line, N-N, depends on the length of the base line. The distance from M_1-$O_1 = \frac{B\,k}{c}$, in millimeters, where B is the length of the base line, c is the unit of the scale of the map, and k is one of the numbers 5, 10, $\frac{25}{2}$, $\frac{50}{3}$, 20, 25, $\frac{100}{3}$, 50. In order to get a favorable point of intersection, S, on N-N and L, such a value in chosen for k as will bring the line, N-N, near the upper border of the drawing area. The distance calculated for $\frac{B\,k}{c}$ is set off on T_1, and N-N is drawn. In using the given values for k, each interval of the divisions on S_1, S_2, ... S_n, is equivalent to a change of the parallax, a, of 20, 10, 8, 6, 5, 4, 3, and 2 hundredths of a millimeter. In Fig. 15, $k = \frac{50}{3}$, and each scale division represents a parallax value of 0.06 mm. Accordingly, beginning with the zero point on M_1-O_1, the parallax scales are numbered as for instance, in Fig. 15 for B.

After these preparations the position of a point, P, can be determined. Set L_1 to a parallax mark on N-N, the point of intersection, S, of the ruler with N-N, and draw G-G perpendicular to M_1-O_1, by using the triangle. Then place L_1 on the reading for x belonging to P. The intersection, P, of the ruler with G-G is the desired point.

The difference in elevation, y, between M_1 and the point, P, is then determined. Set L_1 at the reading of the y-scale belonging to P, mark R of the ruler, L_1, with G-G, and measure R-$Q = y$, by compass or glass scale. The absolute elevation, H, of the point, P, can be determined as done in the figure by a few auxiliary lines parallel to M_1-O_1 directly on the drawing. Another means of determining H directly is the millimeter division on the edge of the

steel triangle. The elevations are noted on a slip of paper beside the division, and the ruler, L_2, is placed so that the divisions in all positions of the triangle passing along L_2 are cut by the axis, M_1-O_1.

Sometimes the ordinates are better calculated if the progress of the work in plotting with graphical determination of the elevation is slower than that of the comparator readings, and an extra man can be engaged on it. Of course, one could try at once to construct the contours on these tracing paper "point sheets", but it is preferable to prick off all these points from the various single sheets into the general map. Certain portions of the ground may be covered from several standpoints. Occasionally, the points from different sheets may supplement and check each other; others can be dispensed with. In any case it is good, in working out many stereograms of difficult country, to plot the photographic base lines beforehand on the triangulation map with all the angles covered.

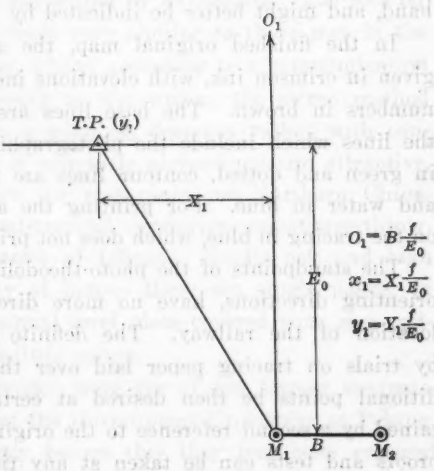


FIG. 16.

The observer using the comparator consults the map before starting work on a new pair of plates, takes out the co-ordinates of the control points, assures himself which parts of his section are shown on his plate alone and which by others once or twice. Of course, it is not quite easy, especially if the observer has not been in the country, to recognize these areas from the orthogonal projection of the map in the comparator, with its limited field of view. Here the new reflecting stereoscope could be used with great advantage.

Characteristic points in a doubly or triply covered section are extremely well suited for special control checks of base lines.

As mentioned previously, it is not absolutely necessary to observe the two plates simultaneously, as the balloon mark of the left, and afterward that of the right, microscope can be successively set on a point; but it is highly desirable that the observer be able to see

stereoscopically, as he is thereby enabled to place the mark as conveniently as though he were able actually to wander at will through the country represented in the photographs.

In photographic surveying no special methods of drawing the contours are involved, as they are filled in from the points obtained, just as from those obtained by any method of surveying. In a preliminary railway survey one usually has to deal with a strip of country from 300 to 700 m. in width, but, in general topographic and geological surveys one has to measure extended areas. For the territory of the most probable line, the greatest accuracy, relatively, is necessary, but, for the more distant country, only a rough approximation is needed. Such supplementary work can be done in the field by free-hand, and might better be indicated by dotted lines.

In the finished original map, the auxiliary triangulation net is given in crimson ink, with elevations indicated in red, and the control numbers in brown. The base lines are drawn in solid green lines; the lines which include the photographic angle of each exposure are in green and dotted, contour lines are in brown, buildings in black, and water in blue. For printing the auxiliary lines may be drawn on the tracing in blue, which does not print out.

The standpoints of the photo-theodolite, the base lines, angles, and orienting directions, have no more direct interest after the definite location of the railway. The definite line is determined as usual by trials on tracing paper laid over the original map. Should additional points be then desired at certain places, they may be obtained by a second reference to the original photographic plates. Also proofs and tests can be taken at any time when doubts arise. This is a very great advantage over the ordinary tacheometric method, where a revision of the map means an entire remeasurement in the field. It is even possible to seek the single points of the located line in the office by using the plates, as the map gives the parallax and the lateral deviation. For a certain value of a or x, it is only necessary to set the balloon marks on the ground in order to obtain directly the ordinate required for the plotting of the profile. In this way all inaccuracies, produced by the more or less free ideal design of the contours, are avoided.

The possibility of plotting on a larger scale certain places, where perhaps structures are to be erected, depends on whether the length

of the base line is sufficient. Cross-sections, also, can be easily taken. Finally, it is possible to transfer points of the line backward from the map into the photographic plates or prints, so that, in the pictures one may follow the proposed line over hill and valley.

THE STEREO-PHOTO-GRAMMETRIC SURVEY OF THE VALLEY NEAR ICHANG.

The foregoing explanations are given in great detail on account of the newness and possible importance of the subject. They deal with the general working plan, the theory of stereo-exposures, the instruments used, and the methods. Something may now be added about the survey of the rocky valley near Ichang.

Not far from Ichang commence the famous gorges of the Yangtze Kiang, noted among travelers, but feared by navigators. The locality and conditions for the survey were very similar to those met in the building of the Bagdad Railway, where, according to a communication from Ph. Holzman and Company to the writer, the stereo method was also used. The Ichang is a winding, deep-cut valley, with canyons and gorges. The scenery is extremely picturesque and attractive, especially to one arriving from the vast plains of Northern China. The slopes are clothed with semi-tropical vegetation, sometimes thinly, and sometimes with dense masses of foliage. Needle oak and high ferns abound. In the vicinity of houses there are single palms, in the bottom of the valley occasional level plots covered with rice paddies, and on the flatter slopes a little cotton.

The collaborators of the writer were six of his former students, just graduated from his class at the Government University at Peking, and speaking English. Shortly before this they had been engaged with the writer on another (but smaller) survey by the photo-stereographic method. These men acted as party heads. The three German engineers were not acquainted with the method. The ten students from Hupeh did not speak a European language, and were not acquainted with German instruments in general, or with the stereographic method.

Unfortunately, the writer could make the survey only during his vacation, namely, in July and August. These months are in every respect the most unfavorable ones in the Yangtze Valley, which, even at the best, has a bad reputation climatically. The heat is perhaps not considerably higher than it may be occasionally in North China,

but the greater humidity causes greater discomfort. Even with a clouded sky, the atmosphere is extremely sultry. In this glowing valley it is practically impossible for a European to work in the direct sun between the hours of 11 A. M. and 3 P. M. Frequent cloudbursts swell the river so that it is impossible to reach the field stations.

The whole party found quarters in a temple, where the photographic plates were developed and worked out. Provisions were difficult to obtain, as there was no fresh meat except chickens, and no vegetables. The muddy water from the rice fields caused ailments of the digestive organs. Nevertheless, serious troubles were prevented by the use of tinned provisions and by filtration of the water.

The most serious and unpleasant drawback was the effect of the heat and moisture on the photographic process. Work in the dark-room could be carried out successfully only during the cooler morning hours, between 2 A. M. and sunrise. The Chinese photographer was not very experienced, and worked slowly. The transport of the good and only recommendable Agfa Chromo Isolar plates was unexpectedly delayed, and the party had to start the work and make trials with many kinds of ordinary dry plates, none of which proved very successful, as some were old and all were improperly packed for the climate. There were some periods of from 6 to 10 days when the sky was clouded. Nevertheless, exposures were made in order to keep things moving. The negatives obtained, however, were so flat that the measurements were made with difficulty. After the arrival of the Agfa plates the negatives were uniformly exceedingly clear and sharp. The emulsion had a much finer grain and seemed to be more heat-resisting. From each plate several prints taken at the left standpoint were made, on which to mark out the points, etc. The new model of the stereo-comparator is equipped with a device for making these marks automatically. A pair of pictures is used for viewing in the reflecting stereoscope.

The negatives, diapositives, and prints were kept in double plate envelopes, especially made for that purpose and printed as field note books. They were made of tracing cloth, and served to protect the plates from damage. The printed record form on each envelope contains all the observations in the field and all the calculations necessary for working out the plates. It is advisable to write the field observations directly on these envelopes, in order to avoid possible confusion in transferring notes.

The theodolite at the disposal of the party was a small light instrument, with short focus (about 100 mm.) and small circle. It was not intended for preliminary railway surveys, but for explorers on general topographic work. In the course of the work it was discovered that the leveling screws were too small in proportion to the sensitiveness of the bubble tube (10″), the power of the telescope was too small (10x), the springs of the micrometer screws were too weak,

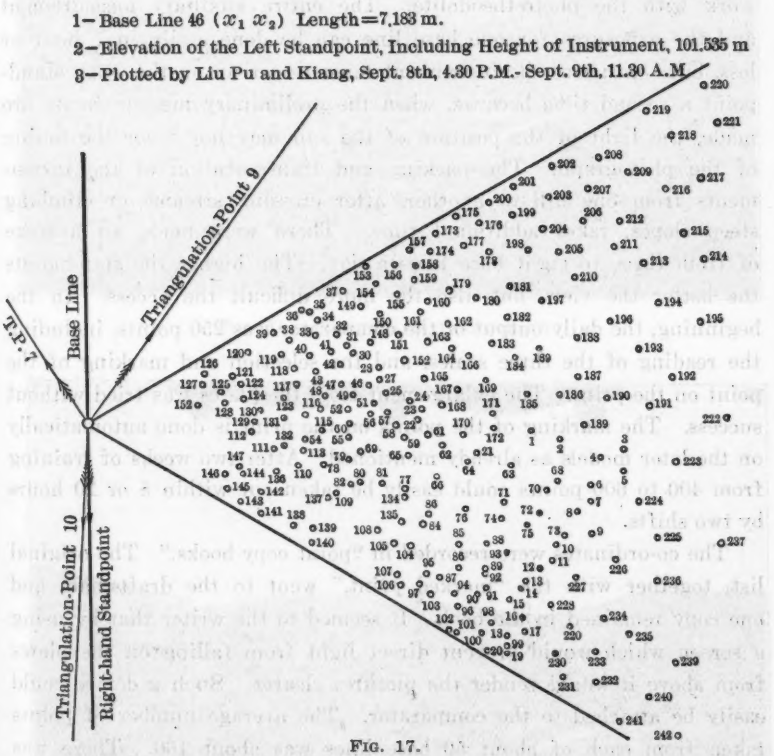


FIG. 17.

and the feet of the tripod too small for a firm setting. Also, the compass did not work satisfactorily. A desirable improvement would be a second bubble tube in the direction of the optical axis as well as a special sighting attachment, such as a direct-vision view-finder. The image on the glass plate is very dark on account of the small stop used. Therefore, the Zeiss Werkstätten proposes, as an apparatus especially suited to railway work on the scale of 1:5 000 to 1:2 500,

a photo-theodolite with $f = 190$ mm., using 13 by 18-cm. plates, with a special device for making exposures turned at 30° from the base line. In this way the image angle can be increased to 110°. In spite of the above-mentioned limitations of the instrument, the accuracy was sufficient for the short distances of the survey, as was proved by the numerous checks.

It is not easy to state the exact time required for the ordinary work with the photo-theodolite. The entire auxiliary measurement and the exposures for one base line can be done easily in 1 hour or less, but sometimes the instrument must be set up on the same standpoint a second time because, when the preliminary measurements are made, the light or the position of the sun may not favor the taking of the photograph. The packing and transportation of the instruments from one hill to another, after crossing streams or climbing steep slopes, takes additional time. There were made an average of from three to eight base lines a day. The higher the standpoints the better the view, but also the more difficult the access. In the beginning, the daily output of the comparator was 250 points, including the reading of the three scales, and the selection and marking of the point on the print. The enlargement from the plates was tried without success. The marking of the points on the print is done automatically on the later models as already mentioned. After two weeks of training from 400 to 600 points could easily be taken out within 8 or 10 hours by two shifts.

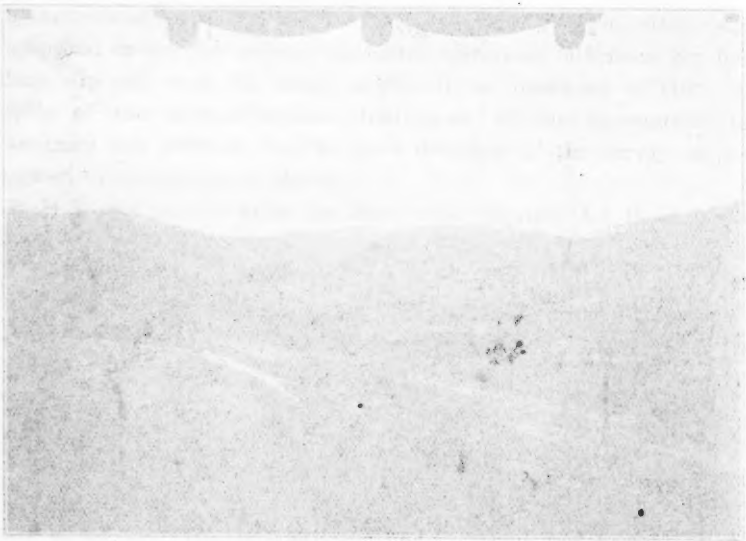
The co-ordinates were recorded in "point copy books." The original list, together with the "marked print," went to the draftsman, and one copy remained in the book. It seemed to the writer that by using a screen which would prevent direct light from falling on the plates from above it would render the pictures clearer. Such a device could easily be attached to the comparator. The average number of points taken from each of about 50 base lines was about 160. There was an average of one point per 25 sq. m. The area of the drawing plate was about 1 sq. m.

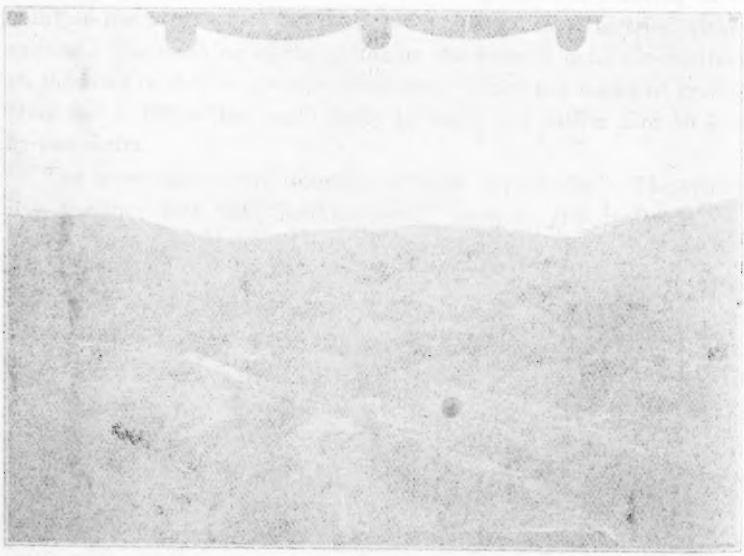
For the scale adopted, 1:2 500, and with a maximum distance of 1 km., a sheet 40 by 40 cm. was required.

After finishing the survey of this rocky valley, work on the less rough country was commenced, and there the conditions were a little different. It was to be mapped on the scale of 1:5 000. It was in-

FIG. 18.—RIGHT-HAND PHOTOGRAPH.

FIG. 19.—LEFT-HAND PHOTOGRAPH.
Base Line 46 (x_1 x_2). Length = 7.183 m.
(From Figs. 18 and 19 the Point Sheet, Fig. 17, was Plotted.)





tended to make from all base lines further exposures with the optical axes turned at 30°, for the purpose of trying out this scale with the now more experienced staff and with the new plotting method, recently published by Dr. Pulfrich. It would have been very valuable indeed to continue in this country (which also could have been surveyed tacheometrically) with the newly arrived fresh and uniform plates and during the cooler autumn weather. Unfortunately, the writer was obliged to return to Peking earlier than he intended, in order to resume his lecture work. The entire result of the work consisted of the following collection of data.

(1) Level notebooks for the bottom of the valley and photographic standpoints.
(2) Angle measurements of triangulation and trigonometric levels.
(3) Calculations of triangulation and trigonometric levels.
(4) Point co-ordinate books, with calculated elevations.
(5) "Point sheets" (tracing paper, 40 by 40 cm.) with "point prints."
(6) Supplementary tacheometric notes.
(7) Photographic material; for each base line three double envelopes containing respectively, negatives, diapositives, and prints.
(8) Finished map, and prints from it.

The whole material was collected and filed in an especially designed chest of drawers, Fig. 21, so that reference and revision could be effected readily.

Conclusions.

The result of this first trial in applying stereoscopy to preliminary railway surveys in China can be considered as satisfactory and encouraging, notwithstanding that the instruments were not quite suitable, that the staff was inexperienced, and that the climate was unfavorable. Although the territory could have been surveyed by tacheometry, it would have required seven times as long. The greatest advantage is that it is not necessary to enter the whole of the territory surveyed, and that the outdoor work takes relatively little time. Later, the working out of the plates for this survey could have been done at Hankow, where there were cool rooms, ice, and electric fans. An improvement for future work might be the use of a larger theodolite,

the greater weight of which would not be significant in the Orient where cheap coolie labor is to be obtained.

The costliness of the necessary precise photographic instruments is a relatively insignificant item of the capital required for the construction of a great railway, especially in China. The saving in capitalized interest by any acceleration of the construction more than justifies such an expenditure. For a like reason, even very expensive apparatus, such as Pulfrich's "stereo-cartograph," might be very advantageously utilized. This instrument plots the selected points automatically, through an ingenious system of levers connecting the scales of the comparator with a marking apparatus on the map.

The von Orel-Pulfrich "stereo-autograph" goes one step farther, as it enables the observer to move the balloon mark along the lines of equal elevation in the view, and at the same time plots the contour lines automatically.

Both these instruments have this great advantage, that they not only make possible the working out of "normal" stereograms, but also of those with equally turned axes, or even with convergent axes, with equal facility. This, as mentioned previously, is not possible in ordinary plotting. These convergent photographs are of special value, as they present the greatest stereoscopic field. If this latter apparatus (price 25 000 marks = $6 250 gold) could have been used, the whole work described herein could have been done in a fraction of the time, by one man instead of four, and more accurately. In the transfer of the points from the point sheets, and in the interpolation of the contours, for example, there are sources of errors to which the plotting engine is not subject.

China is a field in which the photogrammetric method is especially suitable on account of its vast areas of mountainous country. In the North these mountains are bold and bare, in the South and central part they are wooded, but so sparsely that the surface of the ground is sufficiently visible. Moreover, the engineer is not provided by the Government with serviceable ready-made maps as in most Western countries.

The railways are beginning to spread from the seaboard across this mountainous country in all directions. The German section, from Hankow to Ichang, is still for the greater part in flat country, only the part herein described being mountainous, but the American section,

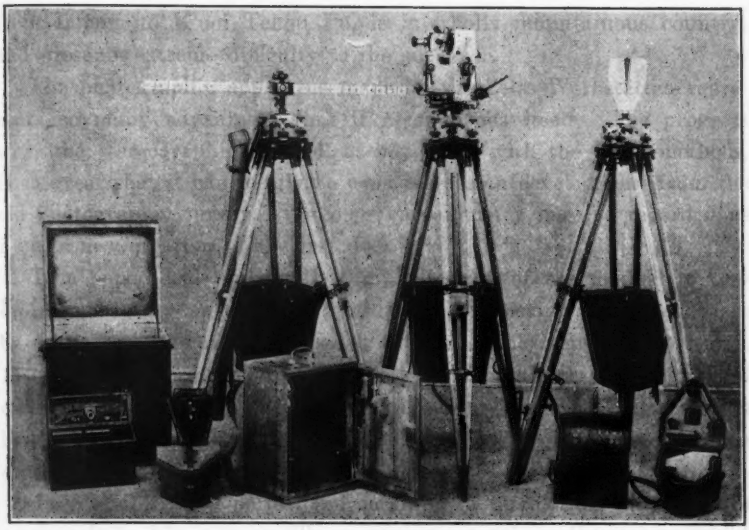
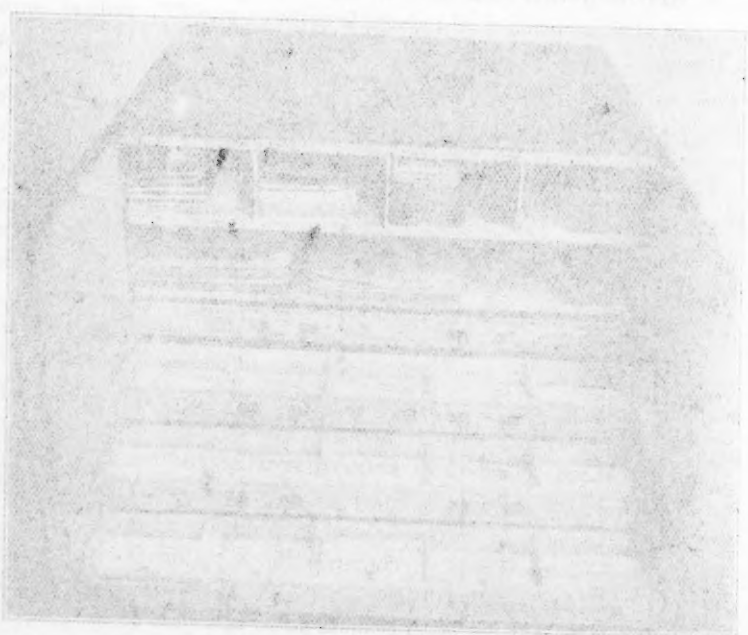


FIG. 20.—EQUIPMENT FOR A STEREO-PHOTO-GRAMMETRICAL SURVEY.

FIG. 21.—CHEST OF DRAWERS FOR STORING MATERIALS USED IN A STEREO-PHOTO-GRAMMETRICAL SURVEY.



from Ichang to K'uei Tchou Fu, is in wholly mountainous country, and presents extreme difficulty to the surveyor.

The photographic apparatus made by the Zeiss Werkstätten represents so many advantages that it recommends itself. The progress with the stereographic method, as compared with the older methods, is so great that it can hardly be considered as a development from the old photographic survey. Photo-stereogrammetry may be regarded as a quite new creation of the last decade.

This paper deals only with the application of the method to preliminary railway survey. With this its engineering possibilities are by no means exhausted. Instead of the simple unrelated pictures, 18 by 30 cm., formerly taken at intervals to indicate the progress of earthwork or structures, stereographic pictures can be taken from a base line fixed once for all. Such pictures would give a much clearer conception of the progress of work.

Besides this, they permit the checking of quantities, statements of earthwork in big cuts or fills, masonry masses, or dimensions which will later be inaccessible, and the agreement of structure to design, and this can be done quickly and accurately at headquarters. The chief engineer of a railway in China will be often unable to visit the field at the most desirable time, on account of the great distances, difficulties of travel, unfavorable climate, etc., and the demand for his presence at headquarters. In such cases the stereoscope will be of especial service, as it will answer his questions accurately. It brings country formerly quite impracticable within the range of an economic survey. The civil engineer frequently has to enter new lands, and will have to make himself familiar with this, the newest branch of surveying practice.

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PAPERS AND DISCUSSIONS

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A METHOD OF DETERMINING STORM-WATER RUN-OFF.

By Charles B. Buerger, M. Am. Soc. C. E.

To be Presented October 7th, 1914.

Synopsis.

The rational method of determining storm-water run-off, as introduced by Emil Kuichling, M. Am. Soc. C. E., and modified in various particulars by designers in the United States and abroad, has usually given satisfactory results. It is not very widely used, however, and the formula methods, of which the Bürkli-Ziegler and the McMath are the most popular, are most generally used, in spite of the common realization of the fact that the results given by them lack consistency, and are very erratic and unreliable. The popularity of these insufficient formulas, notwithstanding their drawbacks, indicates that a formula which will give results reasonably close to those obtained by the rational method will be acceptable for service, even though of somewhat complicated form.

The writer has developed a formula for storm-water run-off of the form

$$q + N q^{\frac{4}{5}} = P$$

in which q is the run-off, in cubic feet per second per acre, and N and P are functions of the variable elements of topography, rainfall,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

etc. Diagrams are given which afford a ready means of obtaining results from this formula.

The writer compares the results obtained by this formula with the gaugings in Rochester by Mr. Kuichling, the Birmingham gaugings by Mr. Lloyd Davies, the Evanston gaugings by C. C. Saner, Assoc. M. Am. Soc. C. E., the New York gaugings by Rudolph Hering, M. Am. Soc. C. E., the St. Louis observations by Robert E. McMath, M. Am. Soc. C. E., the Chicago observations by C. D. Hill, M. Am. Soc. C. E., and others; and notes the variations and discrepancies between observations and calculated results by the new formula. He considers that the results given by his formula are reasonably near to those of the gaugings, and that they may be considered an acceptable equivalent to those determined by the cut-and-try rational method.

Introductory.

At the present time much thought is being devoted to the question of storm-water drainage. Many large cities, such as New York, Chicago, St. Louis, Cincinnati, and others, are studying their storm-water systems, and in some cases are rebuilding where time has proved earlier designs to be inadequate.

Much of the drainage work done throughout the United States, even for large cities, does not have the benefit of such study, but is based on formulas entirely unsuited to the particular local conditions.

On this subject, it has been remarked, with some appearance of justice, that a refinement of methods in the calculation of storm-water run-off is not warranted by any experimental data now available; and it is implied that present methods are good enough when bearing in mind the meagerness and inconsistency of field determinations available to check these methods. It does not appear, however, that the best possible use has been made of existing gaugings.

Existing Methods.

Broadly speaking, two methods of attacking this problem are in customary use: The first is the so-called rational method, with its manifold modifications and variations; the second is the formula method. The rational method was publicly introduced by Mr. Kuichling in 1889.* Mr. August Frühling has given the most complete

* *Transactions*, Am. Soc. C. E., Vol. XX, p. 1.

and satisfactory exposition of this method.* Modified rational methods have been elucidated by Mr. E. E. Wallington Butt,† Mr. Carl H. Nordell,‡ Mr. Maximilian Vicari,§ (the Hauff method), C. E. Grunsky, M. Am. Soc. C. E.,|| Mr. Jean Balcomb,** W. W. Horner, Assoc. M. Am. Soc. C. E.,†† and Messrs. R. O. Wynne-Roberts and T. Brockman.‡‡

The various applications of the rational system differ much in detail of use, but they all follow the system of reproducing, by cut-and-try methods, the existing conditions, to the point of finding corresponding sewer grades, sizes, and assumed rainfall curve and rain absorption or retention.

It is almost everywhere conceded that the rational system, in any of its forms, if used with proper discretion, will give very satisfactory results. It is worth noting, however, that it has not received the widest use, and that formula methods are much more popular. For this there is the reason that the rational system is relatively laborious, and requires, in addition, a material exercise of judgment. Not but that frequent practice will rapidly teach facility of application; but the formula systems have always carried, and probably always will carry, the popular support, because of their convenience and mechanical system of use.

Many run-off formulas have been proposed, among which may be listed:

Hawksley (or Bazalgette):

$$\log. d = \frac{3 \log. A + \log. N + 6.8}{10} \quad \ldots\ldots\ldots\ldots (1)$$

Where d = diameter of sewer, in inches;

N = length of sewer per foot of drop;

and A = area, in acres.

Bürkli-Ziegler:

$$q = C R \frac{S^{0.25}}{A^{0.25}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

* "Handbuch der Ingenieurwissenschaft: Der Wasserbau;" Die Entwässerung der Städte, Leipzig, 1910 (William Engelmann).

† *The Surveyor*, 1907, Vol. XXXII, p. 132.

‡ *Proceedings*, Municipal Engineers of the City of New York, 1909, p. 6, "An Additive Method of Run-off Determination for Storm-Water Sewers."

§ *Gesundheits-Ingenieur*, 1909, Vol. 32, p. 569.

|| *Transactions*, Am. Soc. C. E., 1909, Vol. LXI, p. 496, "The Sewer System of San Francisco, and a Solution of the Storm-water Flow Problem."

** *Proceedings*, Western Society of Engineers, 1910, Vol. XV, p. 699, "The Design of Storm-water Drains in a Modern Sewer System."

†† *Engineering News*, 1910, Vol. 64, p. 326, "Modern Procedure in District Sewer Design."

‡‡ *Canadian Engineer*, 1913, Vol. 24, p. 437.

where q = run-off, in cubic feet per second per acre;
C = an empirical constant;
S = slope, in feet per 1 000 ft.;
A = area, in acres;
and R = rainfall, in inches per hour.

Adams:

$$q = C\,R^{0.83}\,\frac{S^{0.083}}{A^{0.167}} \qquad (3)$$

McMath:

$$q = C\,R\,\frac{S^{0.20}}{A^{0.20}} \qquad (4)$$

New York diagrams (Hering):

$$q = C\,R\,\frac{S^{0.27}}{A^{0.167}} \qquad (5)$$

W. C. Parmley, M. Am. Soc. C. E.:

$$q = C\,R\,\frac{S^{0.25}}{A^{0.167}} \qquad (6)$$

C. E. Gregory, Assoc. M. Am. Soc. C. E.:

$$q = 2.8\,\frac{S^{0.187}}{A^{0.14}} \qquad (7)$$

$$q = \frac{\frac{105\,C}{L}}{84\,\sqrt[5]{A\,S^2}} + 25 \qquad (8)$$

When L = greatest length of sewer, in feet.

All these formulas have this feature in common, that they were devised to represent the conditions at some particular locality, and within certain limiting ranges of slope, area, length, rainfall, and imperviousness. There is no reason to doubt that, for the conditions they were formed to meet, they gave results approximating the facts. Some of them, notably the Bürkli-Ziegler and the McMath, have found a wider application than is reasonably warranted.

An engineer can use a formula with some confidence if sufficient experience is available to assure him that his constants are the proper ones and that he is applying it within its allowable range. In a city, say, like New York, where any formula has been successfully used (and that implies that resulting construction has been neither too large nor too small), and where the conditions of use being con-

sidered are within the range of previous practice, the engineer can be satisfied to design additional storm sewers on the same basis; but, unfortunately, it is only occasionally that it can be determined whether or not the design has been a proper one, from an engineering standpoint.

A great many of the designs for storm-water sewers do not come in this class. They are made for localities where there is little precedent to serve as a guide. The attempt to use a formula of the ordinary design has here at times resulted disastrously. The data of topography and rainfall, of course, are at hand, but, in a new territory, the proper selection of the constants is a matter of much difficulty. For instance, in the exponential formula (McMath type), Cleveland has used a value of $R = 4.0$; New York a value of 2.75; and Chicago a value of 1.0. The rainfall in these cities does not differ materially, and in no way accounts for this variation in the assumed R.

It is not necessary at this late date again to point out the limitations and discrepancies of any of the various formulas used. These are generally realized. In spite of this, it is probable that the great bulk of storm-water drain design is still being executed on the basis of these discredited methods. It is not unusual to read an account of a storm-water drainage system in which all details of construction have been studied most carefully, and where materials, slopes, grades, strength, and other factors are fully worked out. The basis of design —the determination of the needed sizes—will be dismissed in a short paragraph stating that the Bürkli-Ziegler or McMath formula was used, with certain constants; and it is likely that, for a mountainous rocky district of small size, the same constants may be used as for a flat beach town of large extent. It seems to be established that the general preference is for the use of some kind of a formula, and that a reasonably correct and comprehensive one should be an acceptable substitute for the ordinary rough approximations.

There is a need for a formula of general application which can be used in a new locality with some reasonable assurance that it will be fairly appropriate, will be applicable for large areas or small, flat or steep slopes, and heavy or light rainfalls. The results given by such a formula must be compared fairly with the best of experimental information of storm-flow gaugings, and appear to be an envelope

where q = run-off, in cubic feet per second per acre;
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of the bulk of the gauging results, rather than a mean line through them, in order to show a fair margin of safety.

It is not to be expected that such a formula can be a very simple expression; but this sort of complication is not a heavy deterrent to general use, if the results command confidence, as is evident by the general use of the Kutter formula.

The writer has developed such an empirical formula for storm-water run-off, and it is presented herein, together with a detailed comparison with all published gaugings available to him.

The Writer's Run-Off Formula.

q = the run-off, in cubic feet per second per acre;

L = length of longest sewer run, in feet;

C = a coefficient, the ratio of rainfall eventually reaching the sewers to the total rainfall;

A = area, in acres;

S = average slope of longest sewer, L, in feet per 1 000 ft.;

m = average slope of ground surfaces to sewer inlets, in feet per 1 000 ft.;

K = inches of rainfall per hour, lasting for a period of 1 hour, which sewers are to discharge.

$$N = \frac{L}{79 \sqrt[5]{C A S^2} \left(24 + \frac{16}{\sqrt{m}}\right)} \quad \ldots\ldots\ldots\ldots (9)$$

$$P = \frac{3.33 \sqrt{m}\, K C}{\sqrt{m} + 0.87} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$q + N q^4 = P \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Comparison with Run-Off Data.

The validity of a formula of this type rests only on its corroboration by the best of available data. A full coincidence, of course, cannot be expected. The data are scanty, not consistent in themselves, and not always such as to be entirely credible; but it is necessary that the formula be compared with the information we have for all known widely varying conditions.

KUICHLING'S ROCHESTER GAUGINGS.*

Mr. Kuichling's data are fairly full, but very far from being complete. It is necessary at times to estimate certain values as the probable ones, and where that is done a note to that effect is made.

District IV.—Size, about 4 800 by 1 200 ft.; population, 32 per acre; area, 128.67 acres; average street grades, 1 in 130; sewer grades, 1 in 50 to 1 in 630; time to flow through longest sewer, 18 min.; time to reach sewer, 8 min.

$A = 128.67$;
$L = 5\,000$ ft. (estimated);
$m = 7.7$;
$S = 2$;
$C = 0.34$;
I, for time of concentration (26 min.), from Kuichling's Plate I, $= 1.55$;
K, from Kuichling's Plate I, $= 0.87$.

District I.—Area irregular, 356.94 acres; about one-half of the area has a population of 35 per acre; remainder, agricultural and thinly settled; grade of sewered streets, 1 in 150; sewer grades, 1 in 47 to 1 in 910; time to flow through sewers, 34 min.; time to reach sewers, 10 min.

$L = 9\,300$ ft. (estimated);
$m = 6.7$;
$S = 1.4$ (estimated);
$C = 0.25$;
I (for 44 min.) $= 1.0$;
K (as before) $= 0.87$.

District X.—Long, narrow strip; area, 25.12 acres; population, 40 per acre; surface grades, 1 in 172; sewer grades, 1 in 70 to 1 in 330; time to flow through sewers, 10 min.; time to reach sewers, 6 min.

$L = 2\,000$ ft. (estimated);
$m = 5.8$;
$S = 4.0$ (estimated);
$C = 0.39$;
I (for 16 min.) $= 1.78$;
K (as before) $= 0.87$.

* *Transactions*, Am. Soc. C. E., 1889, Vol. XX, p. 1.

District IX.—Shape not known; area, 132.96 acres; population, 36 per acre; street grades, 1 in 151; sewer grades, 1 in 54 to 1 in 400; time to flow through sewers, 15 min.; time to reach sewers, 8 min.

$L = 4\,000$ ft. (estimated);
$m = 6.6$;
$S = 3.0$ (estimated);
$C = 0.37$;
I (for 23 min.) $= 1.6$;
K (as before) $= 0.87$;

District XVII.—Shape unknown; area, 92.27 acres; population, 35 per acre; street grades, 1 in 240; sewer grades, 1 in 100 to 1 in 350; time to flow through sewers, 16 min.; time to reach sewers, 8 min.

$L = 3\,500$ (estimated);
$m = 4$;
$S = 3.0$ (estimated);
$C = 0.36$ (estimated);
I (for 24 min.) $= 1.6$;
K (as before) $= 0.87$.

Table 1 gives the comparative results of gaugings and computed run-offs.

TABLE 1.

District No.	Highest gauging. Q (cubic feet per second).	Kuichling. Rational method.	Buerger formula.
IV.....	71.3	67.5	51.0
I.....	77.01	92.5	78.5
X.....	21.04	17.4	16.0
IX.....	46.0	78.4	71.0
XVII.....	28.45	53.0	46.5

The comparison in Table 1, of highest gaugings and calculations based on the assumed rainfall curve, is not entirely responsive. A detailed comparison of the special storms is given in Table 2.

For this purpose, it is assumed that K can be determined by the relationship,

$$I = \frac{80\,K}{20 + t} \qquad (12)$$

where I is the intensity for a duration of t minutes,

$$\text{or } K = I(0.25 + 0.0125t) \qquad (13)$$

TABLE 2.—KUICHLING'S GAUGINGS.

Date, 1888.	Maximum intensity of rainfall.	Duration of rain at maximum intensity.	K, Estimated.	District I. Q Buerger.	District I. Q Gauging.	District I. Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	District IV. Q Buerger.	District IV. Q Gauging.	District IV. Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	District X. Q Buerger.	District X. Q Gauging.	District X. Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	District IX. Q Buerger.	District IX. Q Gauging.	District IX. Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	District XVII. Q Buerger.	District XVII. Q Gauging.	District XVII. Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	Percentage, average by storm.
May 9th	1.815	35	0.90	78.5	77.01	102	41.2	39.7*	.....	9.3	9.51*	.....	40.0	28.99*	.....	26.6	17.97*	.....	106.8
	1.00	35	0.69	.....	.....	.....	.....	.....	.....	6.8	7.94	96	30.5	24.86	123	18.5	10.84	170	210.0
May 26th	1.00	13	0.53	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	0.75	35	0.41	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
June 24th	1.55	30	0.97	86.5	40.72	210	20.6	33.3	62	24.0	21.04*†	.....	106.0	46.0*†	77	70.0	28.45*†	153.5	153.5
	2.62	20	1.31	117.0	58.82	199	77.0	71.26	108	6.8	7.09	96	29.2	37.5	104	18.5	27.66	67	70.0
June 28th	0.80	20	0.40	28.5	40.72	70	20.6	29.54	70	5.5	8.01	69	22.6	22.09	104	15.6	13.57	115	104.3
July 11th	0.76	15	0.33	25.0	19.93	195	16.7	15.44	108	.....	.....	.....	29.2	19.88	147	18.5	12.71*†	145	166.0
Aug. 4th	1.00	12	0.40	28.5	16.54	169	20.6	12.75	163	12.6	10.01	126	56.0	38.23	146	37.0	28.45	.....	166.8
Aug. 16th	1.616	15	0.71	64.0	27.15	236	41.2	25.93	159	8.0	6.17	130	37.1	21.12	176	25.0	10.94	228	176.4
Aug. 17th	1.388	10	0.5	48.0	25.90	167	27.0	14.94	181	18.4	21.04*†	.....	77.0	46.0*	.....	55.3	28.45*†	.....	150.0
Aug. 26th	2.50	14	1.01	89.0	35.39*	.....	59.1	39.28	150	.....	.....	.....	.....	.....	.....	18.5	16.14	115	96.0
Sept. 16th	0.47	50	0.41	28.5	33.25	84	20.6	28.12	89	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

* Intensity roughly estimated.

† Sewers ran under head; figures given are for maximum discharge without head, or previous to surcharge.

Lloyd Davies' Birmingham Gaugings.*

District I.—Moseley Street.—Shape unknown; area, 312.5 acres; population, 125 per acre; ground slope, 1 in 60; sewer grade, 1 in 85; length, longest line of sewers, 9 565 ft.; time to flow through sewers, 16.75 min.; time to reach sewers, 1.25 min.

$m = 16.7$;
$S = 11.8$;
$C = 1.00$ (estimated);
I (for 18 min.) $= 1.1$.

District II.—Charlotte Road.—Shape unknown; area, 232 acres; population, 4 000; number of buildings, 500; street and road area, 10%; impermeable, 18%, increasing to 42% as the permeable area becomes saturated; length of longest line of sewers, 4 454 ft.; time to flow through sewers, 9.4 min.; time to reach sewers, 2.6 min.; sewer grades, 1 in 55.

$m = 20$ (estimated);
$S = 18.2$;
$C = 0.18$ (estimated);
I (for 12 min.) $= 1.8$.

Table 3 gives a comparison of these gaugings and calculated results, K being determined in the same way as for the Kuichling gaugings.

TABLE 3.

Date, 1904.	District I.				District II.				Average percentage for storm.
	K.	Q.		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	K.	Q.		Ratio: $\frac{Q\text{-}B}{Q\text{-}G}$	
		Buerger.	Gauging.			Buerger.	Gauging.		
Jan. 10th..	0.155	62.5	95.0	66.0	.....	.....	.....	.....	66.0
May 27th..	0.345	172.0	248.0	69.5	0.27	32.5	42.0	77.5	73.5
July 26th..	0.41	203.0	261.0	77.9	0.44	51.0	63.2	80.9	79.5
Aug. 5th..	0.22	93.7	60.1	156.0	0.27	32.5	29.6	110.0	133.5

* *Minutes of Proceedings*, Inst. C. E., 1906, Vol. CLXIV, p. 41.

SANER'S EVANSTON GAUGINGS.*

TABLE 4.—DAVIS STREET SEWER.

Date, 1912.	Time of concentration, in minutes.	Rate of rainfall, in inches.	Area, in acres.	L, in feet, estimated.	C.	S, estimated.	m, estimated.
June 4th...	15	0.60	141	5 900	0.20	5	5
July 3d....	10	2.10	100	5 000	0.20	8	8
July 21st...	45	0.15	420	10 200	0.20	2.5	3
May 28th..	45	0.32	420	10 200	0.20	2.5	3
Aug. 20th..	10	0.78	100	5 000	0.20	8	8

Table 5 gives a comparison of calculated and observed run-off.

TABLE 5.

Date, 1912.	K.	q, in cubic feet per second per acre.	Q, in cubic feet per second.	Q, by gauging.	Ratio: $\frac{Q\text{-Buerger}}{Q\text{-Gauging}}$
June 4th.....	0.25	0.075	10.6	11.8	90.0
July 3d	0.79	0.24	24.0	27.0	89.0
July 21st.....	0.12	.....	....	10.0	....
May 28th.....	0.26	0.03	12.6	30.0	42.0
Aug. 20th.....	0.27	0.08	8.0	5.6	143.0

HERING'S NEW YORK GAUGINGS.†

Sixth Avenue Sewer.—Area = 221 acres; population = 171 per acre; time to flow through sewers = 15 min.

$m = 7$ per 1 000;
$S = 3.5$ (estimated);
$L = 4\,800$ ft.;

Surface, 96.64 acres roof area = 43.5%;
103.32 " paving = 46.5%;
21.76 " grass = 10%;
$C = 0.76$ (estimated).

**Journal*, Western Society of Engrs., 1913, Vol. XVIII, p. 698.
† *Transactions*, Am. Soc. C. E., 1907, Vol. LVIII, p. 468.

Table 6 contains the data of the heavier storms used for comparison.

TABLE 6.

No.	Date.	Maximum rainfall, in inches.	Duration of maximum rainfall, in minutes.
2.....	Dec. 28th, 1887	0.149	4
18.....	June 26th, 1888.....	0.583	13
19.....	June 28th, 1888.....	0.118	4
21.....	July 19th, 1888.....	0.394	10
23.....	Aug. 4th, 1888.....	0.590	10
25.....	Aug. 21st, 1888.....	0.396	10
26.....	Sept. 11th, 1888.....	0.200	10
27.....	Sept. 13th, 1888.....	0.299	7
28.....	Sept. 19th, 1888.....	0.410	16
35.....	Nov. 8th, 1888.....	0.187	5

Table 7 gives a comparison of computed and gauged run-off.

TABLE 7.

No.	K.	Q.		Ratio:
		Computed (C.B.B.)	Gauged.	
2.....	0.67	199	64.3	310.
18.....	1.11	354	200*	177.
19.....	0.53	166	180.9*	92.2
21.....	0.88	277	147.7	187.0
23.....	1.33	398	200*	200.
25.....	0.88	277	200*	138.5
26.....	0.45	143	138.8	103.0
27.....	0.87	277	146.6	189.0
28.....	0.69	199	157.4	127.0
35.....	0.70	199	200	100.0

* The original table does not agree in various columns.

McMath's St. Louis Observations.*

The information in Mr. McMath's paper is far from being accurate enough to receive much weight. The notes as to overcharge of sewers are not more than observations. It is most difficult to estimate what is intended by the expression "often overcharged." The writer has assumed in the following comparison that a value $K = 1$, corresponding to full discharge in a period of about $1\frac{1}{2}$ years, meets this condition. In all cases, $C = 0.75$, and $m = 15$.

Table 8 gives a comparison of Mr. McMath's records and the calculated run-off.

* *Transactions*, Am. Soc. C. E., 1887, Vol. XVI, p. 179.

TABLE 8.

Location.	*A.*	*L*, estimated.	*S*, estimated.	Required *Q*.		Actual Capacity, Calculated.	
				McMath.	Buerger.	*Q.*	Remarks.
Compton Avenue (2)..	284	6 800	8.	330	425	253	Often overcharged.
Camp Spring (1).....	55	3 000	8.7	87	99	100	" " *
" " (6).....	516	9 100	13.	524	770	733	" " *
" " (2).....	49	2 800	11.	80	93	86	" " *
Biddle Street (2).....	82	3 600	4.	120	122	60	" "
" " (5).....	441	8 400	6.	462	619	496	" " *
" " (6).....	458	8 600	6.	475	641	605	" " *
" " (7).....	657	10 200	5.5	640	850	1 165	" " *
Northwestern (1).....	113	4 300	9.4	154	171	102	" "
" (2).....	138	4 700	10.	183	221	178	" "
Cass Avenue (1).....	61	3 100	2.3	96	92	49	" "
" " (3).....	94	3 900	6.	134	141	151	" " *
" " (5).....	190	5 500	5.5	237	284	180	" "
Benton Street (2).....	155	5 000	7.	200	248	163	" "
Grand Avenue (3).....	138	4 700	6.	182	220	122	" "
" " (4).....	198	5 600	5.5	246	307	138	" "
" " (5).....	236	6 100	5.	283	341	266	" "
" " (6).....	271	6 600	5.	312	392	325	" "
" " (7).....	304	7 000	5.	348	440	256	" "
Elliot Avenue (1).....	31	2 200	15.	55	56	52	" "
Broadway Branch (1).	33	2 300	7.8	58	59	40	" "
" " (2).	46	2 700	9.	77	83	43	" "
" " (3).	51	2 900	10.	83	89	55	" "
" " (4).	66	3 200	11.	102	116	90	" "
Salisbury Street (1)...	49	2 800	19.	81	88	60	" "
" " (2)...	91	3 800	12.	131	159	98	" "
Ferry Street (4).....	338	7 400	11.	373	506	274	" "
Trudeau Street (1)....	120	4 400	16.	163	204	112	" "
Arsenal Street Branch (1)..	36	2 400	13.6	62	65	50	" "
Arsenal Street Branch (2)..	49	2 800	12.	82	88	48	" "

* Mr. McMath attributes overcharge to local conditions, rather than to insufficient capacity.

Hill's Chicago Observations.*

Mr. Hill's observations resemble Mr. McMath's in that the record of results is vague, and consists only of a statement of the adequacy or inadequacy of the sewer.

In considering the information given, allowance must be made for a later statement by Mr. Hill,† from which the following is quoted:

"Since the paper under discussion was written, there have been a number of severe rainstorms, that have again demonstrated the inadequacy of many of Chicago's sewers, and have severely tested others that were previously above suspicion. To some extent the opinions of the writer have been modified. On the evening of May 24th there was a downpour aggregating nearly two inches, with a maximum intensity of three inches per hour during a period of fifteen

* *Journal*, Western Society of Engrs., 1902, Vol. VII, p. 425.

† *Loc. cit.*, p. 442.

minutes. The result of this storm was that nearly every deep basement in the city was flooded."

In Table 9, for the purpose of comparison, the run-off as calculated by the Hill method, as outlined in his paper, is given in the column so marked. The coefficient, C, is estimated from the population and the relatively impervious areas noted. The sewer grade, S, is taken as 0.50, and the street grades, m, as 2.0 per 1 000. K is taken as 1 in.

Water-shed Flood Run-off.

Table 10 contains data relating to the maximum flood discharge of streams in the eastern section of the United States, for water-sheds up to 100 sq. miles in area.*

The flood discharges in Table 10 are plotted on the diagram, Fig. 1, by their numbers, together with two curves drawn according to the proposed run-off formulas for values of K of 1.0 and 1.5, respectively.

For comparison, the same assumptions are made throughout.

$L = 300\ \sqrt{A}$, in acres;
$C = 0.15$;
$m = 15$;
$S = 10$;
$K = 1.0$ and 1.50.

Comparison with Other Computations.

Walworth Sewer, Cleveland, Ohio.†—The data and computations used by Mr. W. C. Parmley are taken from Table 2, page 347, of his paper on this sewer. From Fig. 1, on page 351, the highest rainfall, lasting 1 hour, and corresponding to $R = 4$, is taken as $K = 1.1$. From the rainfall records, the indications are that this sewer will never be more than filled. This, of course, is much more liberal than ordinary design, which permits of periodic flooding.

Table 11 gives comparative results of the methods used.

St. Louis Sewers.

Table 12, giving comparative results of the rational method by Mr. W. W. Horner, and the proposed formula, is made up from an article by Mr. Horner.‡

*These are from data by Mr. Kuichling in the New York Barge Canal Report, 1901, p. 852 *et seq.*, and *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 650 *et seq.*

† *Transactions*, Am. Soc. C. E., 1905, Vol. LV, p. 341.

‡ "Modern Procedure in District Sewer Design," by W. W. Horner, *Engineering News*, 1910, Vol. LXIV, p. 328.

TABLE 9.

	Location of Sewers.	Area, in acres.	Population per acre.	Impervious area.	C (Estimated.)	Run-off, in Cubic Feet per Second per Acre.		Sewer capacity, in cubic feet per second per acre.	Observation.
						Hill.	Buerger.		
A1	West Adams St., West of River	266	70	0.90	0.54	0.20	0.35	0.20	Adequate.
A2	Armitage Ave., West of River	3 280	40	0.30	0.34	0.07	0.11	0.08	Adequate as yet.
A3	Adams St., East of River	30	1 500	1.00	1.00	0.86	0.36	1.56	Few complaints.
A4	Addison St., East of Robey St	300	30	0.30	0.26	0.19	0.17	0.147	Adequate.
A5	Augusta St., West of River	196	120	0.90	0.75	0.21	0.56	0.26	
B1	Bryn Mawr, West of Lake Michigan	275	10	0.20	0.12	0.083	0.065	0.12	Adequate.
B2	Belmont Ave., West of Lake View Ave	85	60	0.50	0.50	0.264	0.44	0.20	Basements flooded.
C1	North Clark St., North of River	400	90	0.80	0.64	0.18	0.51	0.22	Adequate.
C2	Clybourne Place, East of River	180	90	0.40	0.34	0.135	0.26	0.16	"
C3	Chicago Ave., West of River	48	100	0.90	0.70	0.76	6.98	0.33	"
C4	Chicago Ave., East of River	108	120	0.90	0.75	0.264	0.92	0.25	"
D1	Dearborn St., Randolph to Madison	1.75	1 500	1.00	1.00	1.74	1.81	1.71	Few complaints.
D2	Division St., West of River	367	100	0.90	0.70	0.18	0.63	0.14	Doubtful.
D3	Division St., East of River	50	10	0.40	0.12	0.31	0.11	0.33	Adequate.
D4	Diversey Boulevard, East of River	675	30	0.40	0.26	0.10	0.17	0.113	Doubtful.
E1	Erie St., West of River	33	100	0.90	0.70	0.84	1.05	0.50	Adequate.
F1	S. 45th Ave., North of River	3 800	40	0.30	0.34	0.07	0.12	0.76	Undeveloped.
F2	Foster Ave., West of Lake Michigan	265	10	0.20	0.12	0.083	0.062	0.12	Adequate.
F3	Fullerton Ave., East of River	550	60	0.60	0.50	0.17	0.33	0.11	Doubtful.
F4	Fullerton Ave., East of Ashland Ave	500	70	0.60	0.54	0.17	0.40	0.064	Frequently flooded.
F11	Fullerton Ave., East of Sheffield Ave	200	70	0.70	0.54	0.213	0.48	0.085	" "
F5	56th St., West of Lake Michigan	*1 100	20	0.30	0.20	0.09	0.08	0.036	" "
F6	51st St., West of Lake Michigan	900	20	0.30	0.20	0.094	0.080	0.066	Barely adequate.
F7	45th St., West of Lake Michigan	500	40	0.40	0.34	0.11	0.20	0.058	Frequently flooded.
F8	41st St., West of Lake Michigan	410	50	0.40	0.40	0.11	0.26	0.07	" "
F9	41st St., West of Prairie Ave	98	40	0.40	0.34	0.16	0.34	0.17	
F10	Fullerton Ave., West of River	1 010	30	0.50	0.30	0.15	0.17	0.10	
G1	Granville Ave., West of Lake Michigan	425	10	0.20	0.12	0.076	0.05	0.14	Adequate.
G2	Graceland Ave., West of Lake Michigan	160	10	0.10	0.12	0.09	0.07	0.088	Few complaints.
G3	Garfield Boulevard, West of Michigan Ave	90	20	0.30	0.20	0.16	0.18	0.21	
G4	Garfield Boulevard, East of Michigan Ave	155	20	0.30	0.20	0.14	0.13	0.14	Complaints due to outlet.
H2	Harrison St., West of River	260	80	0.90	0.60	0.20	0.51	0.20	Adequate.
I1	Indiana Ave., South of Garfield Boulevard	45	20	0.30	0.20	0.20	0.20	0.20	
I3	Irving Park Boulevard, South of Robey St	203	30	0.30	0.26	0.13	0.19	0.18	Adequate.
L1	Lawrence Ave., West of Lake Michigan	370	10	0.30	0.12	0.11	0.07	0.122	"
L2	Lake Ave., North of 44th Place	26	50	0.40	0.40	0.35	0.47	0.25	Back-water from F7.

* Includes 240 acres of parks.

TABLE 9.—*(Continued.)*

	Location of Sewers.	Area, in acres.	Population per acre.	Impervious area.	C (Estimated.)	Run-off, in Cubic Feet per Second per Acre.		Sewer capacity, in cubic feet per second per acre.	Observation.
						Hill.	Buerger.		
L3	Lawrence Ave., East of Robey St.	162	20	0.30	0.20	0.14	0.14	0.16	Adequate.
L4	Lawrence Ave., West of River	1 300	3	0.30	0.10	0.09	0.08	0.065	Undeveloped.
L5	Lake View Ave., North of Roslyn Place	7.5	50	0.40	0.40	0.48	0.86	0.36	Too small.
M1	Michigan Ave., River to Van Buren	58	500	0.90	0.90	0.29	1.18	0.20	No complaints.
M2	Montrose Ave., West of River	1 450	5	0.10	0.10	0.06	0.08	0.08	Undeveloped.
M3	Michigan Ave., South of Garfield Boulevard	54	20	0.30	0.20	0.19	0.20	0.13	
M4	Montrose Ave., East of Robey St.	170	20	0.30	0.20	0.14	0.15	0.10	Adequate.
M5	Madison St., West of River	280	90	0.90	0.64	0.20	1.03	0.185	"
N1	North Ave., West of River	340	80	0.90	0.60	0.19	0.54	0.25	"
N2	North Ave., East of River	380	80	0.90	0.60	0.18	0.54	0.105	Unknown.
P1	Polk St., West of River	131	120	0.90	0.75	0.24	0.91	0.21	Adequate.
R1	Roscoe St., West of Lake Michigan	170	30	0.40	0.26	0.14	0.21	0.10	Deep basements flooded.
R2	Robey St., North of Montrose Ave.	450	20	0.20	0.20	0.11	0.11	0.077	Undeveloped.
R3	Robey St., North of Byron St.	870	20	0.20	0.20	0.095	0.10	0.068	"
R4	Robey St., North of Roscoe St.	1 260	20	0.20	0.20	0.09	0.08	0.076	"
R5	Robey St., North of Addison St.	1 100	20	0.20	0.20	0.09	0.08	0.054	"
R6	Rush St., North of River	290	90	0.80	0.80	0.19	0.54	0.26	Adequate
R7	Randolph St., West of River	275	60	0.90	0.50	0.20	0.42	0.19	"
S1	Sheridan Road, North of Lawrence Ave.	2 365	10	0.30	0.12	0.078	0.03	0.09	
S2	Sheridan Road, South of Lawrence Ave.	975	25	0.30	0.23	0.09	0.13	0.10	
S3	Surf St., West of Lake View Ave.	50	30	0.50	0.26	0.30	0.30	0.16	
S4	70th St., West of Yates Ave.	1 400	5	0.10	0.10	0.06	0.03	0.093	
S5	73d St., West of Railroad Ave.	1 200	5	0.10	0.10	0.06	0.04	0.08	
T1	West 22d St., West of Western Ave.	3 400	60	0.80	0.50	0.10	0.19	0.074	Occasionally flooded.
T2	35th St., West of Lake Michigan	720	60	0.50	0.50	0.15	0.32	0.104	Frequently flooded.
T3	Thorndale Ave., West of Lake Michigan	185	10	0.20	0.12	0.09	0.07	0.135	Adequate.
T4	West Taylor St., West of River	182	150	0.90	0.80	0.22	0.77	0.29	"
T5	West 12th St., West of River	148	120	0.90	0.70	0.23	0.74	0.35	"
V1	Vincennes Ave., North of 51st St.	50	30	0.30	0.26	0.19	0.28	0.16	"
V2	West Van Buren St., West of River	300	70	0.90	0.54	0.19	0.44	0.18	"
W1	Wentworth Ave., North of Calumet R.	5 500	5	0.20	0.10	0.05	0.02	0.06	Undeveloped.
W2	Western Ave., North of North Branch	700	5	0.10	0.10	0.07	0.04	0.06	"
W3	Wabash Ave., South of Garfield Boulevard	54	20	0.30	0.19	0.19	0.19	0.31	
W4	Webster Ave., East of River	200	50	0.50	0.40	0.21	0.36	0.20	Many complaints.
W5	Webster Ave., East of Halsted St.	65	60	0.70	0.48	0.28	0.56	0.24	" "

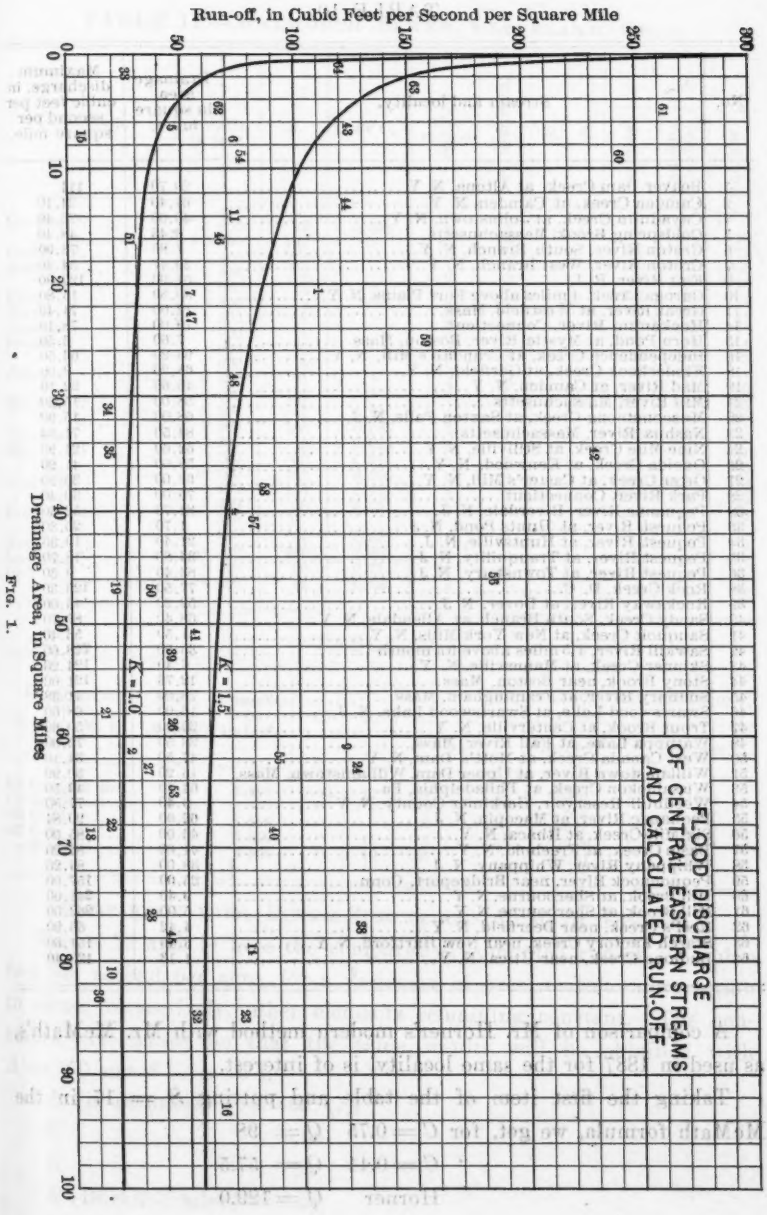

Run-off, in Cubic Feet per Second per Square Mile
0
50
100
150
200
250
300
Drainage Area, in Square Miles
0
10
20
30
40
50
60
70
80
90
100
FLOOD DISCHARGE
OF CENTRAL EASTERN STREAMS
AND CALCULATED RUN-OFF
K = 1.0
K = 1.5

FIG. 1.

TABLE 10.

No.	Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.
1	Beaver Dam Creek, at Altona, N. Y.	20.70	111
2	Camden Creek, at Camden, N. Y.	61.40	24.10
4	Cayadulta Creek, at Johnstown, N. Y.	40.00	72.40
5	Coldspring Brook, Massachusetts	6.43	48.40
6	Croton River, South Branch, N. Y.	7.80	73.90
7	Croton River, West Branch, N. Y.	20.47	54.40
9	Flat River, R. I.	61.00	120.90
10	Garoga Creek, 4 miles above Fort Plains, N. Y.	80.80	15.80
11	Great River, at Westfield, Mass.	14.00	71.40
14	Hockanum River, Connecticut	79.00	78.10
15	Horn Pond, at Mystic River, Boston, Mass.	7.60	6.50
16	Independence Creek, at Crandall's Mill, N. Y.	98.20	66.50
18	Kinderhook Creek, at Garfield, N. Y.	68.20	9.00
19	Mad River, at Camden, N. Y.	46.60	22.10
21	Mill River, Massachusetts	58.00	15.50
22	Musconetcong Creek, at Saxton Falls, N. J.	68.00	15.90
23	Nashua River, Massachusetts	84.50	71.04
24	Nine-Mile Creek, at Stillville, N. Y.	62.60	124.90
26	Oneida Creek, at Kenwood, N. Y.	59.00	41.20
27	Otter Creek, at Caster's Mill, N. Y.	63.00	30.90
28	Park River, Connecticut	76.00	30.40
32	Pequanac River Riverdale, N. J.	84.70	52.50
33	Pequest River, at Hunts Pond, N. J.	1.70	25.30
34	Pequest River, at Huntsville, N. J.	31.40	19.30
35	Pequest River, at Tranquility, N. J.	34.80	18.70
36	Pequest River, at Townsbury, N. J.	83.40	9.60
38	Rock Creek, D. C.	77.50	126.30
39	Rockaway River, at Dover, N. J.	52.20	43.00
40	Sandy Creek, South Branch, at Allendale, N. Y.	68.40	87.70
41	Sauquoit Creek, at New York Mills, N. Y.	51.50	53.40
42	Sawkill River, 4.5 miles above its mouth.	35.00	228.60
43	Skinner Creek, at Mannsville, N. Y.	6.40	124.20
44	Stony Brook, near Boston, Mass.	12.73	121.00
45	Sudbury River, at Framingham, Mass.	78.00	41.38
46	Swartswood Lake, at Swartswood Lake, N. J.	16.00	68.00
47	Trout Brook, at Centerville, N. Y.	23.00	50.60
48	Watuppa Lake, at Fall River, Mass.	28.50	72.00
50	West Canada Creek, at Mott's Dam, N. Y.	47.50	34.10
51	Williamstown River, at Upper Dam, Williamstown, Mass.	16.20	30.90
53	Wissahickon Creek, at Philadelphia, Pa.	64.60	43.50
54	Woodhull Reservoir, Herkimer County, N. Y.	9.40	77.80
55	Pequanac River, at Macopin, N. J.	62.00	90.84
56	Six-Mile Creek, at Ithaca, N. Y.	46.00	185.00
57	Basin Creek, at Freehold, N. Y.	41.00	81.20
58	Whippany River, Whippany, N. J.	38.00	84.20
59	Pequonnock River, near Bridgeport, Conn.	25.00	157.00
60	Mill Brook, at Sherbourne, N. Y.	9.40	241.00
61	Mill Brook, at Sherbourne, N. Y.	5.00	262.00
62	Reel's Creek, near Deerfield, N. Y.	4.42	66.90
63	Starch Factory Creek, near New Hartford, N. Y.	3.40	151.60
64	Budlong Creek, near Utica, N. Y.	1.13	120.40

A comparison of Mr. Horner's modern method with Mr. McMath's, as used in 1887 for the same locality, is of interest.

Taking the first item of the table and putting $S = 17$ in the McMath formula, we get, for $C = 0.75$ $Q = 98$

" $C = 0.44$ $Q = 57.5$

Horner $Q = 129.0$

TABLE 11.—WALWORTH SEWER, CLEVELAND, OHIO.

Sub-main.	*L*, length, in feet.	*A*.	*f* or *C*.	*s*, also called *S*.	*m*, assumed.	*R*.	*K*.	*Q*.		
								Parmley.	Bürkli-Ziegler.	Buerger.
Gordon, N.	1 600	12	0.625	5	9	4	1.1	30	24	18
" S.	5 800	123	"	10	16	....	....	245	164	172
Clark, W.	6 600	350	"	7	9	....	....	540	337	455
Alum, N.	1 300	8	"	6	9	....	....	22	19	13
" S.	4 800	53	"	10	16	....	....	121	87	74
Purdy	1 300	9	"	6	9	....	....	25	20	14
Waverly	1 300	9	"	6	9	....	....	25	20	14
Guernsey	700	8	"	6	9	....	....	22	19	14
Swiss, N.	800	75	"	6	9	....	....	144	100	135
" S.	600	6	"	8	16	....	....	19	16	11
Junction, N.	2 900	71	"	4	9	....	....	124	86	100
" S.	7 200	187	"	7	9	....	....	320	206	235
Burton, N.	2 600	57	0.75	4	9	....	....	123	88	97
" S.	9 800	466	0.625	5	9	....	....	630	375	540
Pollock	1 200	19	0.625	5	9	....	....	44	34	31
Mill, N.	700	42	0.75	4	9	....	....	96	58	106
" S.	13 900	797	0.625	3	9	....	....	858	494	718
Pearl, N.	4 100	154	0.75	4	9	....	....	282	185	245
" S.	2 400	58	0.75	3	9	....	....	117	83	98
Brevier, N.	600	.....	.....	.....	.....	.....	.....	.....	.....	.....
" S.	600	4	0.625	15	25	....	....	14	14	8
Kenilworth	11 150	416	0.625	3	9	....	....	500	303	405
Scranton	2 000	26	0.75	40	49	....	....	115	87	61
Cliff	3 400	32	0.625	3	9	....	....	59	44	42

TABLE 12.

Location of sewer, M. H. to M. H.	Area, in acres.	Length, *L*, in feet.	*C*.	*S*.	*m*.	*K*.	*Q*, IN CUBIC FEET PER SECOND.	
							Horner.	Buerger.
153 to main line..	61.4	3 800	0.44	17	20	1.7	129.0	117.0
175 to 165	3.15	950	"	7	10	"	6.9	5.7
162 to 161	14.9	1 600	"	8	10	"	32.7	25.3
157 to b	33.5	2 600	"	15	15	"	73.8	63.7
c to 155	48.9	3 200	"	19	20	"	105.0	92.9

COMPARISON WITH BÜRKLI-ZIEGLER AND MCMATH RESULTS.

Tables 13 and 14 give the comparative computed run-off, in cubic feet per second per acre, for a variation of area, and for a variation in slope, respectively, other elements remaining constant. The constants are chosen so that the same results will be obtained with $A = 100$ and $S = m = 4$.

L (is assumed) $= 400\sqrt{A}$

$C = 0.50$

$K = 1.0$

R (Bürkli-Ziegler) $= 3.58$

R (McMath) $= 3.03$

TABLE 13.

$S = m = 4$.

Area, in acres.	q = Run-Off, in Cubic Feet per Second per Acre.		
	Bürkli-Ziegler.	McMath.	Buerger.
10	1.42	1.27	1.00
50	0.95	0.92	0.85
100	0.80	0.80	0.80
200	0.67	0.70	0.78
500	0.54	0.58	0.68
800	0.48	0.53	0.62
1 000	0.45	0.50	0.59
2 000	0.38	0.44	0.54
5 000	0.30	0.36	0.46
8 000	0.27	0.33	0.41
10 000	0.25	0.32	0.38

(Slide-rule work.)

TABLE 14.

$A = 100$.

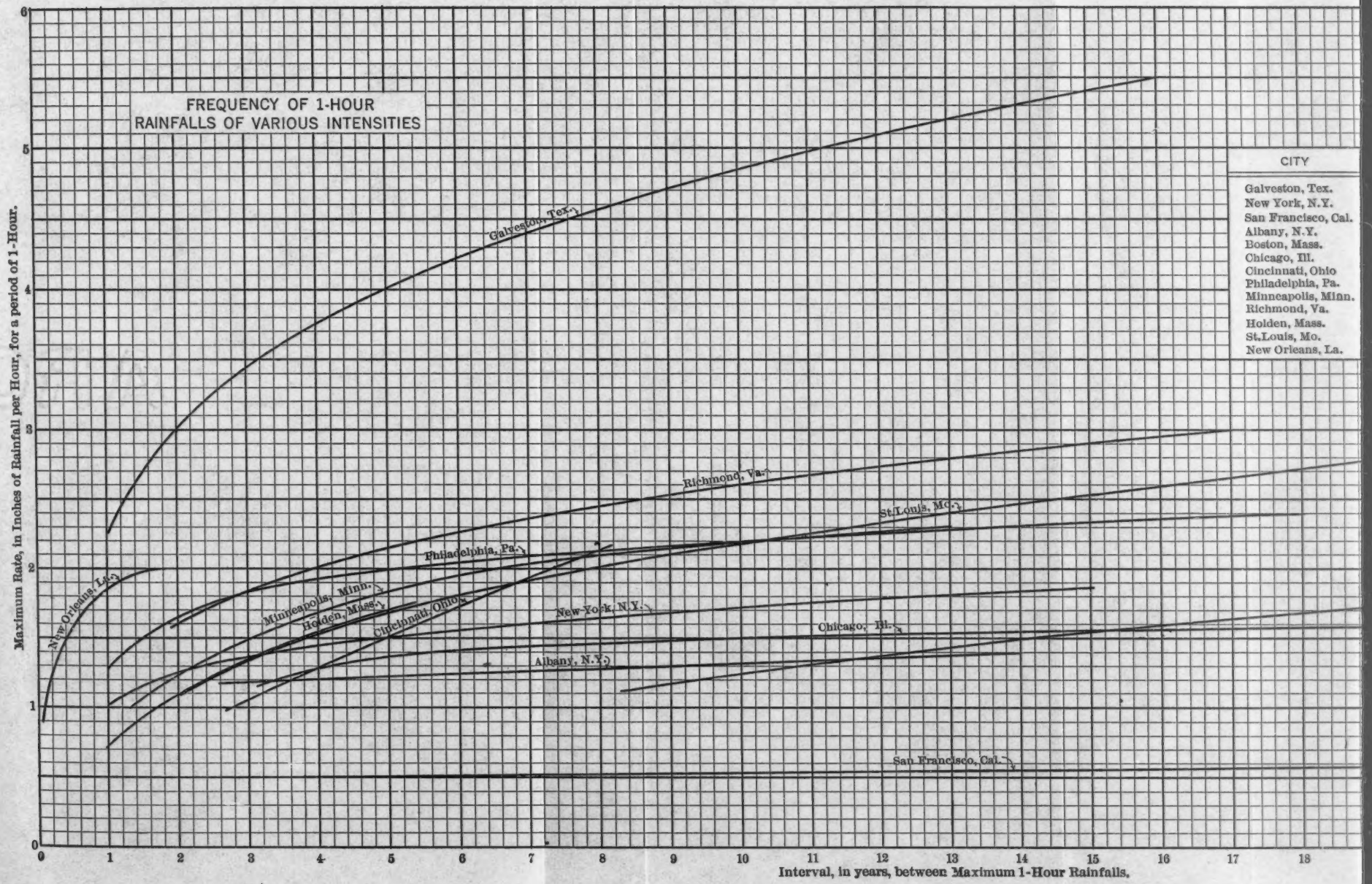
$S = m$	q = Run-Off, in Cubic Feet per Second per Acre.		
	Bürkli-Ziegler.	McMath.	Buerger.
3	0.75	0.75	0.76
4	0.80	0.80	0.80
5	0.85	0.88	0.85
7	0.92	0.88	0.95
10	1.00	0.96	1.00
15	1.11	1.04	1.05
20	1.20	1.10	1.10
30	1.32	1.19	1.20
40	1.42	1.26	1.25
50	1.50	1.32	1.30
60	1.57	1.37	1.35
80	1.69	1.46	1.38

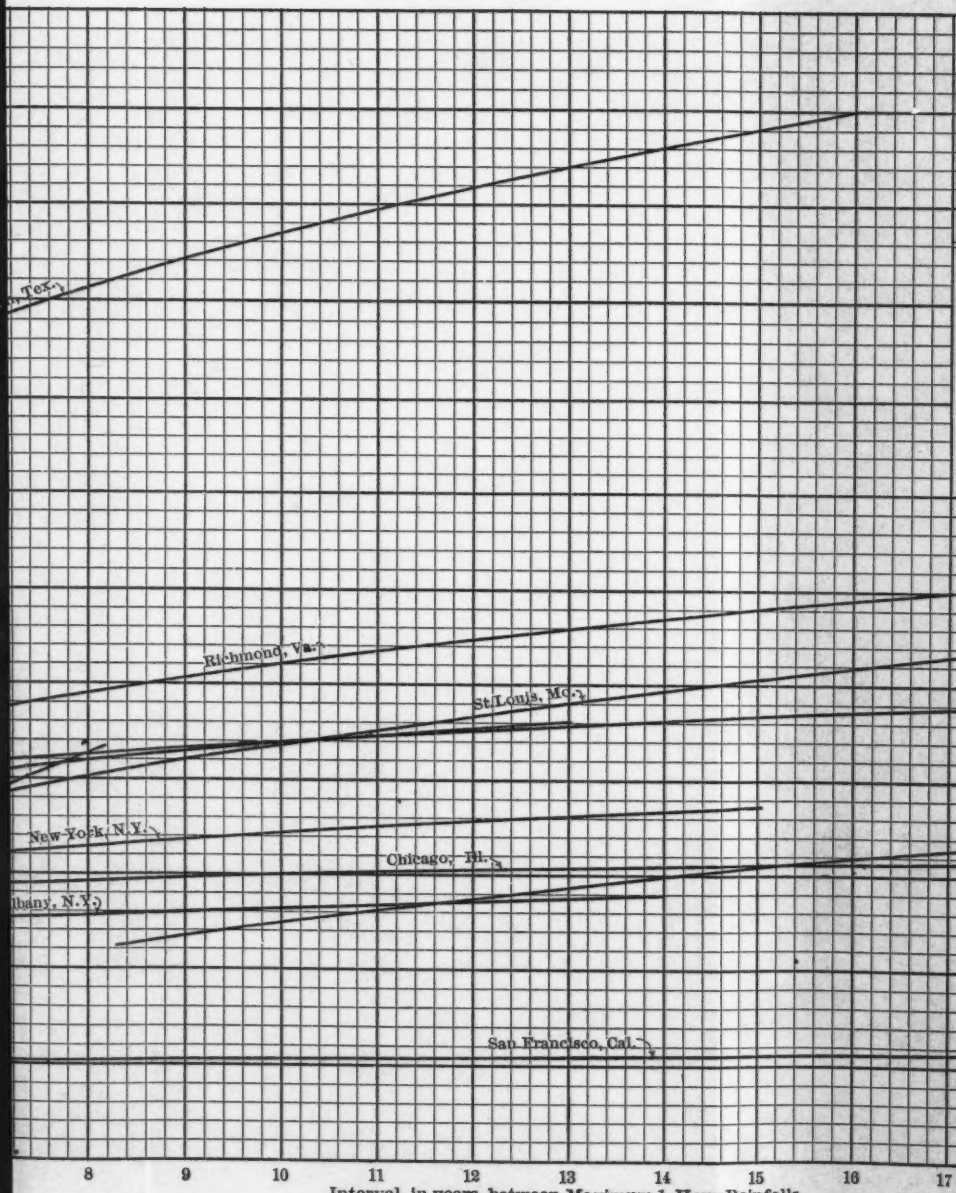
(Slide-rule work.)

Tables 13 and 14 are of use only to those who think in terms of either of these two formulas.

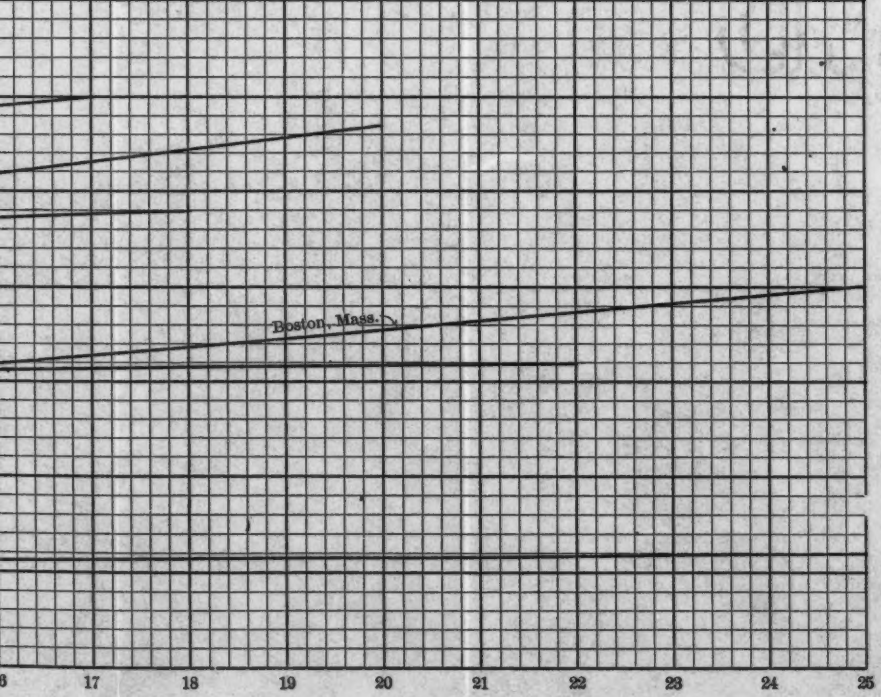
Determination of C.—The value of C, which, in the writer's formula, represents the ratio between the run-off and the total rainfall, or, in other words, the ratio of that part of the rainfall not absorbed by or retained on the soil to the total rainfall, will vary much with special local conditions. A study of the nature of the land and its improvements will give some reasonable conclusions as to C.

The existing improvements are not always to be used as a basis of storm-water drain design in a growing community, and for such a


FREQUENCY OF 1-HOUR
RAINFALLS OF VARIOUS INTENSITIES
Maximum Rate, in Inches of Rainfall per Hour, for a period of 1-Hour.
Interval, in years, between Maximum 1-Hour Rainfalls.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Galveston, Tex.
Richmond, Va.
St. Louis, Mo.
Philadelphia, Pa.
New Orleans, La.
Minneapolis, Minn.
Holden, Mass.
Cincinnati, Ohio
New York, N.Y.
Chicago, Ill.
Albany, N.Y.
San Francisco, Cal.
CITY
Galveston, Tex.
New York, N.Y.
San Francisco, Cal.
Albany, N.Y.
Boston, Mass.
Chicago, Ill.
Cincinnati, Ohio
Philadelphia, Pa.
Minneapolis, Minn.
Richmond, Va.
Holden, Mass.
St. Louis, Mo.
New Orleans, La.


Tex.
Richmond, Va.
St. Louis, Mo.
New York, N.Y.
Chicago, Ill.
Albany, N.Y.
San Francisco, Cal.
8
9
10
11
12
13
14
15
16
17
Interval, in years, between Maximum 1-Hour Rainfalls.

CITY	SOURCE OF INFORMATION		PERIOD	
Galveston, Tex.	*Engineering News*, Vol. 71, Jan. 8, 1914,	p. 69.	1898	1913
New York, N.Y.	Met. Sew. Com. Report 9, Nov. 1913,	p. 8.	1896	1911
San Francisco, Cal.	*Transactions*, Am. Soc. C.E., Vol. 65,	p. 312.		1908
Albany, N.Y.	*Journal*, Western Soc. Engrs., Vol. 18,	p. 672.	1899	1912
Boston, Mass.	" " " " " "	p. 674.	1879	1904
Chicago, Ill.	" " " " " "	p. 675.	1889	1910
Cincinnati, Ohio	" " " " " "	p. 676.	1897	1912
Philadelphia, Pa.	" " " " " "	p. 679.	1895	1912
Minneapolis, Minn.	" " " " " "	p. 677.	1899	1912
Richmond, Va.	" " " " " "	p. 680.	17 years	
Holden, Mass.	*Engineering News*, Vol. 69,	p. 704.	1897	1912
St. Louis, Mo.	" " Vol. 64,	p. 328.	1890	1909
New Orleans, La.	*Engineering Record*, Vol. 54,	p. 641.	1895	1906



section, not fully developed, some estimate must be made of the probable ratio, C, corresponding to an assumed development at some future time.

A fact, not always fully recognized, is that rain-water falling on a fully impervious surface does not of necessity reach the sewers. This is particularly likely to be the case in a moderately populated district. Rain-water falling on roofs will fully run off from these roofs, but it will often be discharged on lawns and other unprotected surfaces, and the net run-off from these roofs will accordingly be little if any greater than if the rain fell on an undeveloped area of the same imperviousness as the surrounding soil. The same is true of rain-water from paved walks, which are usually bordered by grassed earth of large extent compared with the walks themselves, and the discharge on these unprotected surfaces is largely absorbed. For heavily populated districts, arrangements usually permit of direct discharge of roof and pavement water to the sewers, and no such reduction of run-off need be made.

A very weighty factor in the value of C is the perviousness of the soil, and the corresponding run-off from unprotected areas. The theoretical range will run from 0 for light sandy soils to 1.00 for some rocky or clayey soils. For less unusual conditions, and for the ordinary range of practice, the writer assumes a range of from 5% for soils of sandy nature up to 50% for clay soil. Based on these figures and assumptions, as to the type, number, and size of buildings, areas of streets, walks and their character, the curves of Fig. 2 are derived, in which C is expressed as a function of the population per acre. Table 15 gives a comparison of estimated values of C, based on population, from a number of sources. The coefficient, C, probably does not, in all cases, represent identical conception.

Determination of K.—The determination of K, the rainfall lasting for a period of 60 min., is a relatively simple matter. Records for shorter periods are not always available, but 60-min. rainfalls are often to be had. Plate XIII gives the rate of rainfall for 1 hour duration, in a number of cities, plotted as a function of the average interval in years between such rainfalls. Having a curve of this character for any locality, a selection of the frequency with which the drains are to run full will determine the value of K to be used.

TABLE 15.

Population per acre.	Kuichling.*	Butt.†	Grunsky.‡	Imhoff.§	Buerger (average).
10	0.10	0.16	.....	0.10	0.21
20	0.20	0.26	0.30	0.15	0.27
30	0.28	0.34	0.40	0.20	0.33
40	0.38	0.44	0.50	0.25	0.39
50	0.47	0.55	0.55	0.31	0.45
60	.....	0.64	0.60	0.37	0.51
70	.....	0.74	0.65	0.42	0.57
80	.....	0.84	0.70	0.48	0.61
90	.....	0.94	0.73	0.54	0.66
100	.....	1.00	0.75	0.60	0.69

*H. N. Ogden, M. Am. Soc. C. E., "Sewer Design," 1909, p. 62.
† E. E. Wallington Butt, *The Surveyor*, 1907, Vol. 32, p. 133.
‡ C. E. Grunsky, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXV, p. 319.
§ K. Imhoff, "Taschenbuch für Kanalisations-Ingenieure," 1912, p. 17.

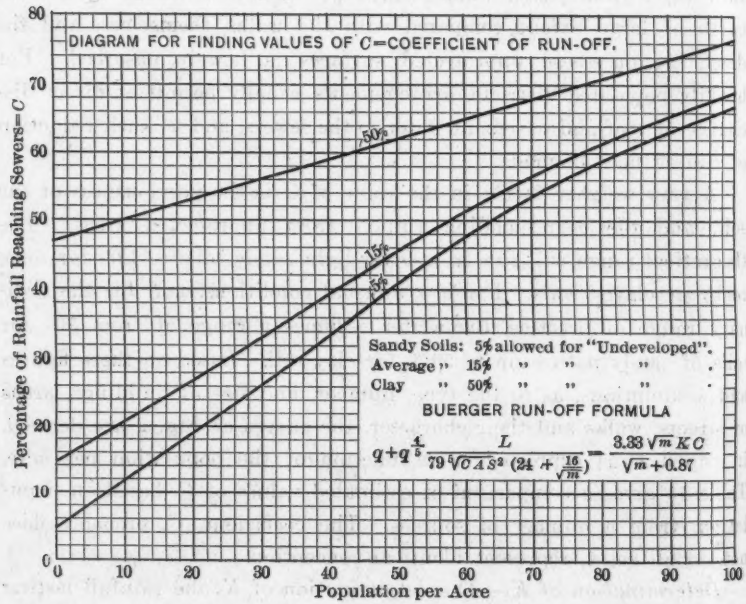
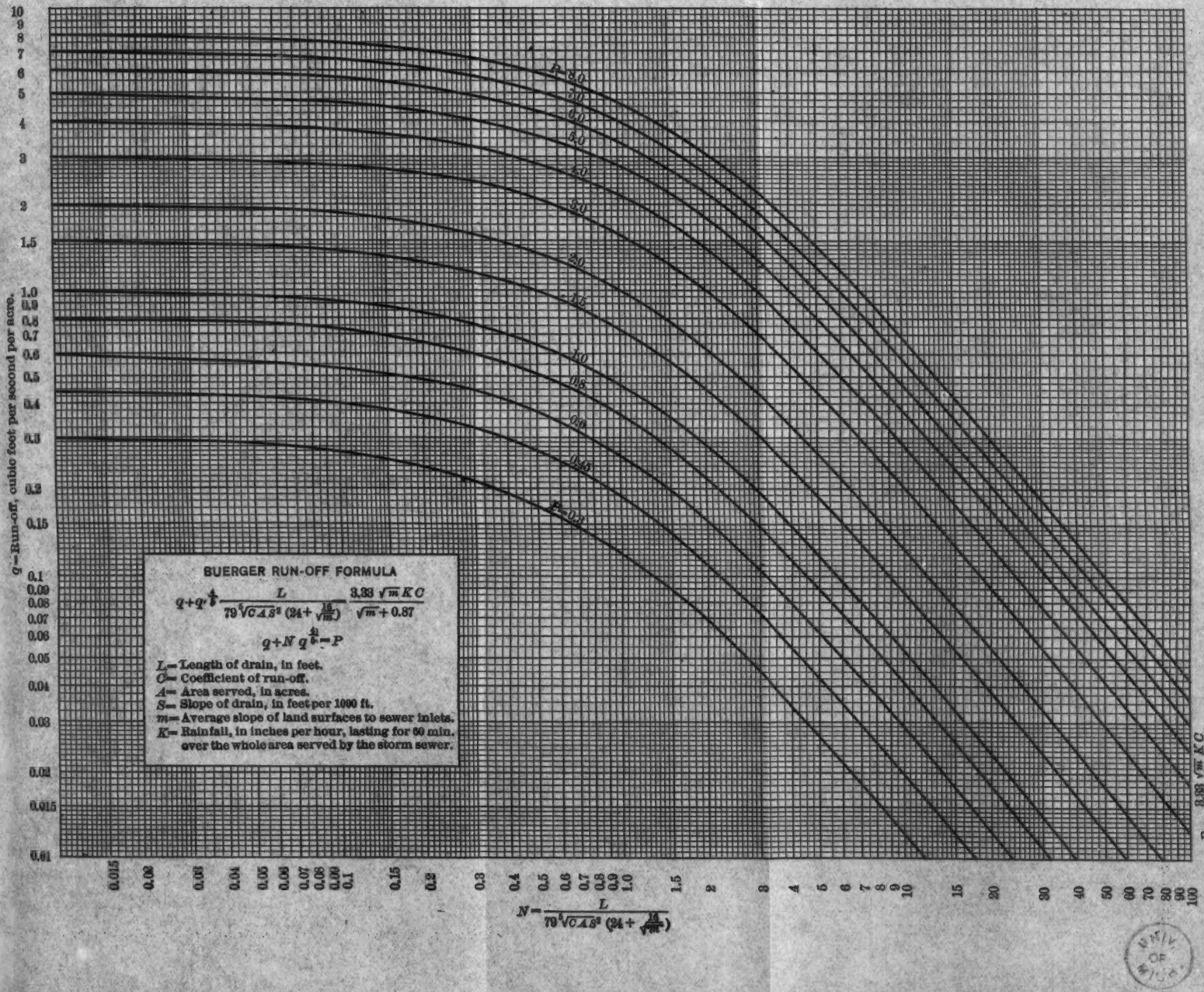


FIG. 2.

Mr. Imhoff suggests* that an assumption that the sewers will be fully charged by the run-off once every year is a reasonably economical and practical basis, and that longer intervals are not warranted. This would correspond to a value of K ranging from 1 to 1.25 for the Central Eastern States, as shown by Plate XIII.

*Loc. cit., p. 19.



BUERGER RUN-OFF FORMULA

$$q+q\cdot\frac{4}{b}\cdot\frac{L}{79\sqrt[5]{CAS^2}\,(24+\frac{16}{\sqrt{m}})}\cdot\frac{3.33\sqrt{m}\,KC}{\sqrt{m}+0.87}$$

$$q+Nq^{\frac{4}{b}}=P$$

L = Length of drain, in feet.
C = Coefficient of run-off.
A = Area served, in acres.
S = Slope of drain, in feet per 1000 ft.
m = Average slope of land surfaces to sewer inlets.
K = Rainfall, in inches per hour, lasting for 60 min. over the whole area served by the storm sewer.

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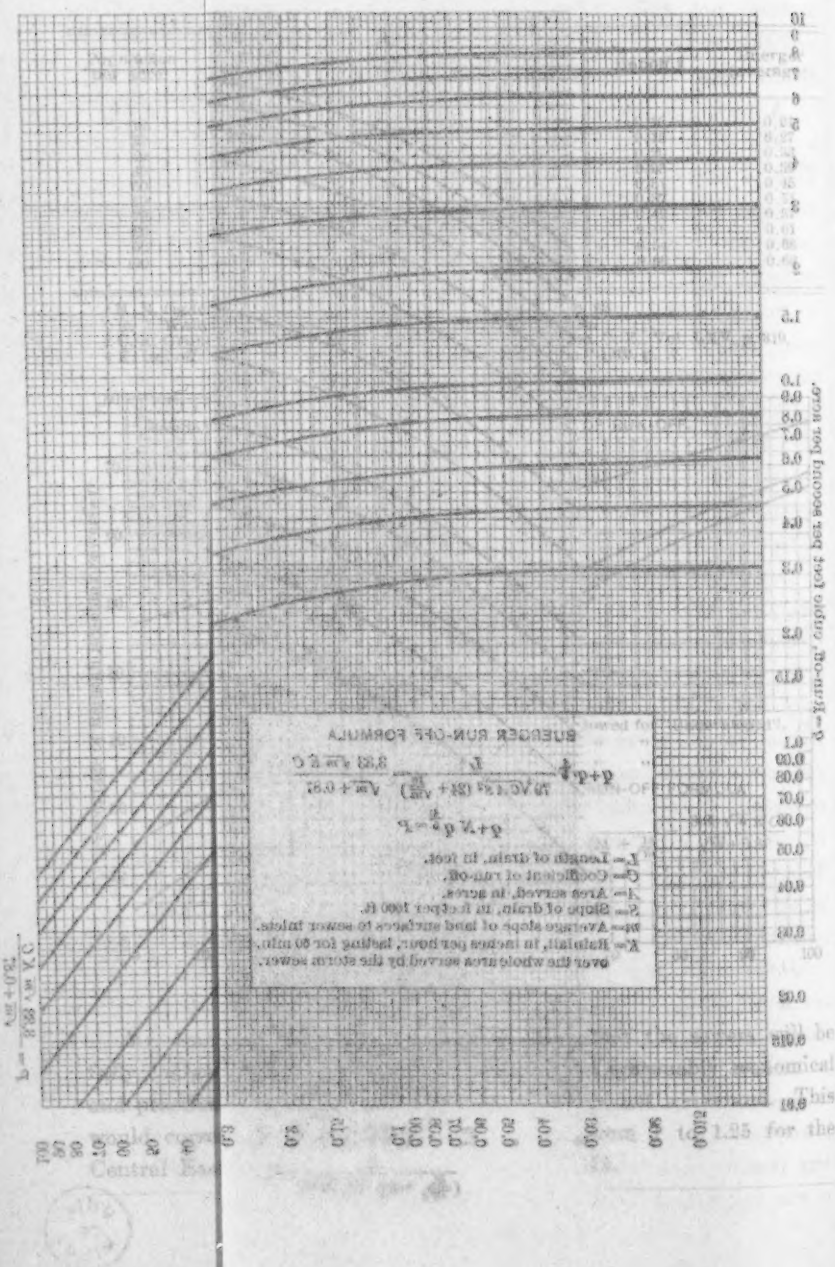

BUERGER RUN-OFF FORMULA
L = Length of drain, in feet.
C = Coefficient of run-off.
I = Area served, in acres.
S = Slope of drain, in feet per 100 ft.
m = Average slope of land surface to sewer inlets.
K = Rainfall, in inches per hour, lasting for 60 min. over the whole area served by the storm sewer.

The rainfall for which the drains are to provide and the intervals between full flow, must be matters of judgment to be decided according to the relative importance of adequate drainage and first cost.

There is no exact basis on which such a comparison can be made. The amount of damage resulting from a flooding of storm sewers cannot be gauged accurately in advance in dollars and cents. In a few cases the loss resulting from any flooding is so obviously heavy that it immediately appears necessary to provide drains of ample capacity to prevent such flooding under the worst possible conditions. For more ordinary cases, the resultant damage is moderate, and, if not so frequent that the annoyance of flooding must enter as a serious factor, it may be better to submit to such occasional losses than to endure the heavier first cost of drains of larger size. Each case must rest on its own merits, and it is not unreasonable in one city to have an allowed interval between full flow in the drains of from 6 months to 20 years for districts of different character.

Solution of Proposed Formula.—After the selection of values for C and K, which requires some degree of judgment, the work of determining the run-off for any districts of known area, sewer length, sewer grade, surface, and slope becomes entirely mechanical.

To aid in the ready solution of the formula, three diagrams are given. The first of these, Fig. 3, gives the value of P, the right-hand member of the equation. The second diagram, Fig. 4, gives the value of N, the second term of the left-hand member. The third diagram, Plate XIV, gives the values of q, as a function of N and P.

Typical Application of Run-Off Formula.

This calculation is made for one district in Altoona, Pa. A storm curve of maximum intensity is assumed (not here reproduced).

$$C = 0.50$$

$$A = 130 \text{ acres.}$$

Rational Method.—Time of concentration:

1 800 ft. of 24-in. sewer on 80-ft. slope,	1½ min.
1 800 " " 54-in. " " 18-ft. "	2 min.
Time to reach sewers.....	5 min.
	8½ min.

Intensity for 8½ min. duration = 6.2 in. per hour.

$Q = 130 \times 6.2 \times 0.5 = 403$ cu. ft. per sec.

Buerger Formula.—

$L = 3\,600$,

$A = 130$,

$C = 0.50$,

$K = 2.3$ (from rainfall curve),

$m = 49$ (from map),

$S = 40$ (" "),

P, from Fig. 3 $= 3.4$,

N, from Fig. 4 $= 0.158$,

q, from Plate XIV $= 3.0$,

$Q = 130 \times 3.1 = 390$ cu. ft. per sec.

DIAGRAM FOR FINDING VALUES OF THE EXPRESSION,

$$P = \frac{3.33\sqrt{m}\,KC}{\sqrt{m}+0.87}$$

IN THE BUERGER RUN-OFF FORMULA

$$q + q^{\frac{4}{5}} \frac{L}{79\sqrt[5]{CAS^2}\,(24+\frac{16}{\sqrt{m}})} = \frac{3.33\sqrt{m}\,KC}{\sqrt{m}+0.87}$$

C=Coefficient of run-off.
m=Average slope of land surfaces to sewer inlets.
K=Rainfall, in inches per hour, lasting for 60 min.

Find value of m at bottom of diagram. Run up this line as indicated by dotted line to curve. At this point run horizontally to K line then vertically to C line, then run horizontally to P values at right hand side.

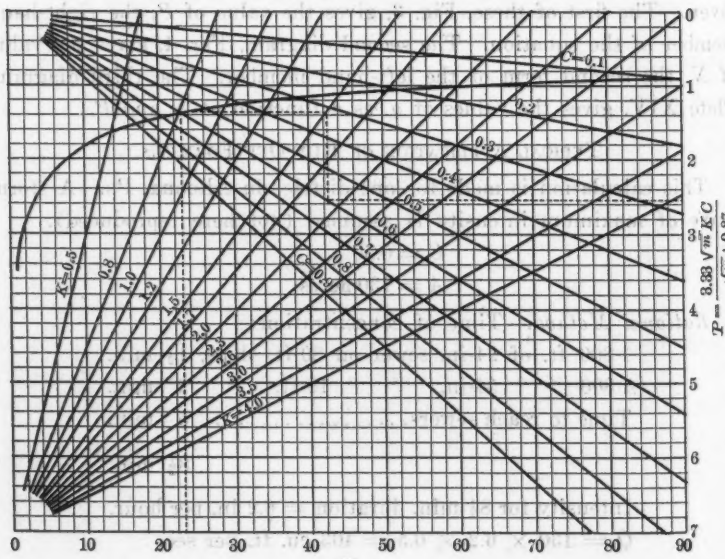


FIG. 3.

DIAGRAM FOR FINDING VALUES OF THE EXPRESSION,

$$N = \frac{L}{79\ \sqrt[5]{C A S^2}\,(24 + \frac{16}{\sqrt{m}})}$$

IN THE BUERGER RUN-OFF FORMULA:

$$q = Q^{\frac{4}{5}} \cdot \frac{L}{79\sqrt[5]{C A S^2}\ (24 + \frac{16}{\sqrt{m}})} = \frac{3.33\sqrt{m}\ KC}{\sqrt{m} + 0.87}$$

L = Length of drain, in feet.
C = Coefficient of run-off.
A = Area served, in acres.
S = Slope of drain, in feet per 1000 ft.
m = Average slope of land surfaces to sewer inlets.

Find value of m at bottom of diagram, run vertically as indicated by dotted line to C curve. At this point run horizontally to S line then vertically to A line then horizontally to L curve, then run up vertically to N values at top of diagram.

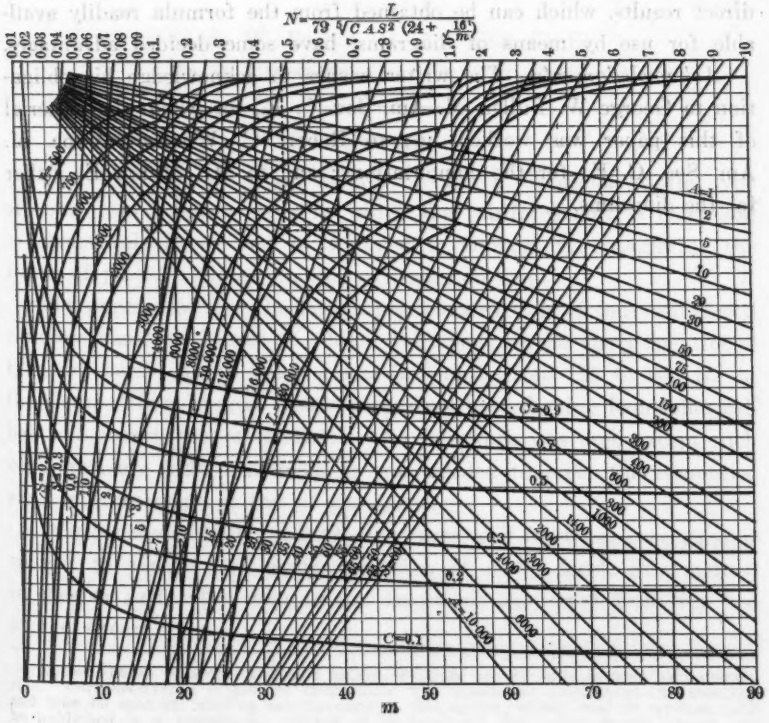


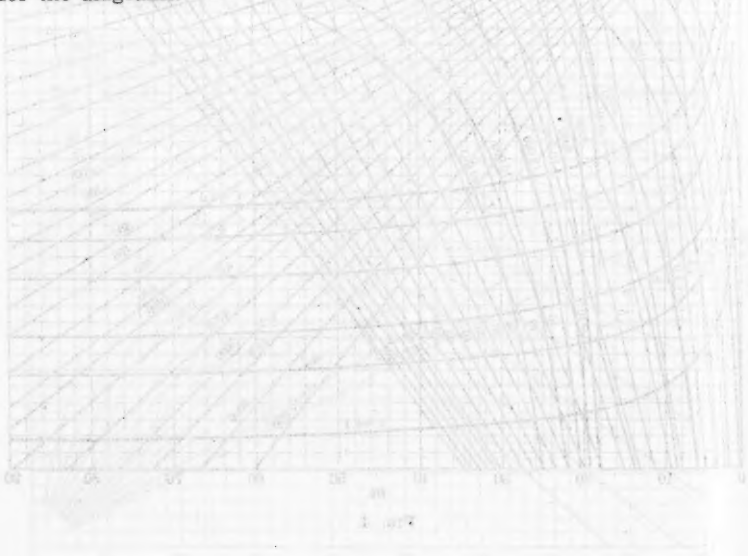
FIG. 4.

The writer has had a large number of computations made on areas varying in imperviousness, extent, length, slope, and with various rates of rainfall, and can state, without qualification or limitation, that the proposed formula gives results in all cases reasonably close to those which will be obtained by the rational method, and that the formula can be taken as an acceptable equivalent.

As compared with the exponential formulas, the results show no relationship except an occasional coincidence at some points under suitable combinations of conditions.

In the writer's judgment the correctness and usefulness of any run-off formula must stand on its corroboration of results obtained by the rational method. The new formula fully meets this condition. Because of its complicated form, there will no doubt be many who will consider a graphical or cut-and-try method more useful. The direct results, which can be obtained from the formula readily available for use by means of diagrams, have some decided advantages.

Acknowledgments.—The writer wishes to acknowledge his obligation to George W. Fuller, M. Am. Soc. C. E., for whom the material of this paper was worked up; to Robert C. Wheeler, Assoc. M. Am. Soc. C. E., for the computations; and to Mr. Myron E. Fuller for the diagrams.



AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS.

By S. W. Bowen, M. Am. Soc. C. E.

To be Presented October 21st, 1914.

Synopsis.

The object of this paper is to bring out a discussion of the type of reinforced concrete arch in which the reinforcement is composed of structural shapes and is designed to support the weight of the forms of plastic concrete in the arch ring on ribs.

The paper describes the design and construction of four reinforced concrete viaducts, in two of which arch spans of the above mentioned type are used. These spans range in length from 125 to 200 ft., in the clear. Three-hinged, ribbed arches are used, having hemispherical, ball and socket, cast-steel hinges. No falsework was used except such comparatively light timbering as was needed to erect the structural steel reinforcement of two of the spans.

It is concluded that, for high structures, and for those over streams subject to sudden and great variation of water level, this method is cheaper and safer than to use falsework supported from the bed of the stream.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The tables give unit and total costs, as well as the cost per linear foot, per square foot of horizontal and vertical projection, and per cubic foot of volume, for each viaduct.

Introduction.

The City of Fort Worth, in Tarrant County, Tex., is traversed by the Trinity River and its branches, the Clear and West Forks. As will be seen by the map, Fig. 1, this stream is very crooked, and cuts the town up badly, necessitating a number of bridges. The city proper lies on the high ground, around which the stream flows. The growth of the residence districts toward the east and west, and the great increase in traffic between the city proper and North Fort Worth, in which the packing houses are located, has recently made necessary the construction of a new viaduct and the reconstruction of three old ones.

The principal feature of this paper is the description of the design and construction of the three-hinged, ribbed-arch spans. In the ribs and rib braces these spans have structural steel reinforcement which is designed to support the weight of the forms and plastic concrete of the ribs and braces during construction. By this means falsework was dispensed with, except such light timbering as was used in erecting some of the structural reinforcement.

These viaducts were built with the proceeds of the sale of bonds issued by the county. The total amount of funds available for construction purposes was about $656 000, exclusive of the accrued interest on the funds in bank.

The firm of Brenneke and Fay, Consulting Engineers, of St. Louis, was selected by the Commissioners' Court to prepare plans and specifications for, and to supervise the construction of, the four viaducts.

The bridges which the viaducts were to replace were old, and too light and narrow to take care of the increasing traffic. The general feeling among the residents of the county was that the new structures should be planned for the future, and be built in such a manner, and of such materials, that they would require the minimum amount of maintenance, and last indefinitely. Therefore, reinforced concrete was selected as the material best suited to the conditions, and was used for all parts of the structures. Creosoted wood block paving

was recommended by the engineers as being light, smooth, and durable, but owing to the high cost of such paving at Fort Worth, vitrified brick was finally selected.

Design.

General Description.—The structures are known, respectively, as the Main Street, West Seventh Street, Samuels Avenue, and East Fourth Street Viaducts. They are mentioned in the order of their cost, the Main Street Viaduct being the largest of the four. The general map, Fig. 1, shows the location of the crossings, and Plate XV shows the four structures drawn to the same scale, so that their relative size can readily be seen.

The widths of roadways and sidewalks are as follows: Main Street, 54-ft. roadway and two 8-ft. sidewalks; West Seventh Street, 43-ft. roadway and two 6-ft. sidewalks; Samuels Avenue and East Fourth Street, 30-ft. roadway and two 5-ft. sidewalks. In each case provision is made for two electric interurban car tracks in the roadway. These tracks are placed in the center of the roadway for the Main and West Seventh Street Viaducts, and at the sides for the two smaller structures. The Main Street Viaduct is the most important of the four, and is on the main artery of travel between the city proper and North Fort Worth. Therefore, it is made wider than the others. The width of roadway is such that four wagons and two street cars can pass abreast. The width of the roadway of the West Seventh Street Viaduct is the same as the width, between curbs, of the street leading up to the crossing. This is ample to accommodate two wagons and two street cars abreast. The two smaller viaducts, being short, and of less importance, as to the volume of traffic to be carried, than the first two named, have roadways of sufficient width to allow only one wagon and two street cars to pass abreast.

From out to out of concrete construction, the length of the Main Street Viaduct is about 1 752 ft., made up of one 225-ft. arch span over the stream, two 175-ft. and one 150-ft. arch spans, one 68 ft. 9-in., two 62 ft. 6-in., seven 50-ft., and two 25-ft. girder spans. The remainder is made up of earth fills enclosed by retaining walls of the semi-gravity type. The West Seventh Street Viaduct is about 1 041 ft. long, consisting of one 137 ft. 6-in. arch span over the stream, and seven 50-ft. girder spans; the remainder consists of retaining

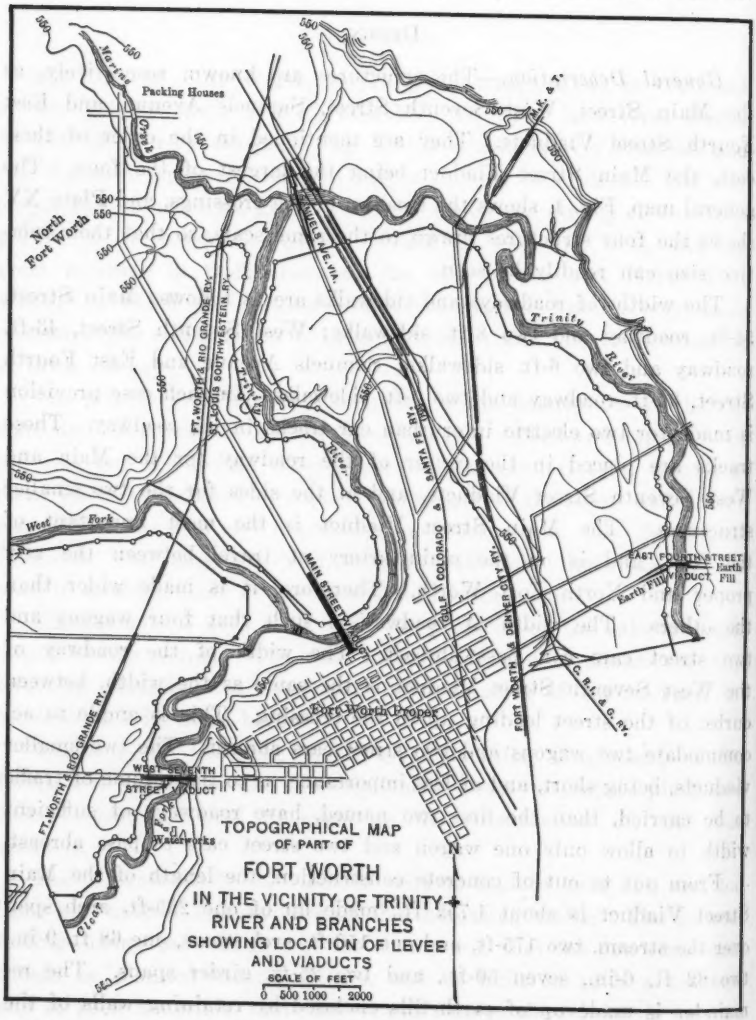
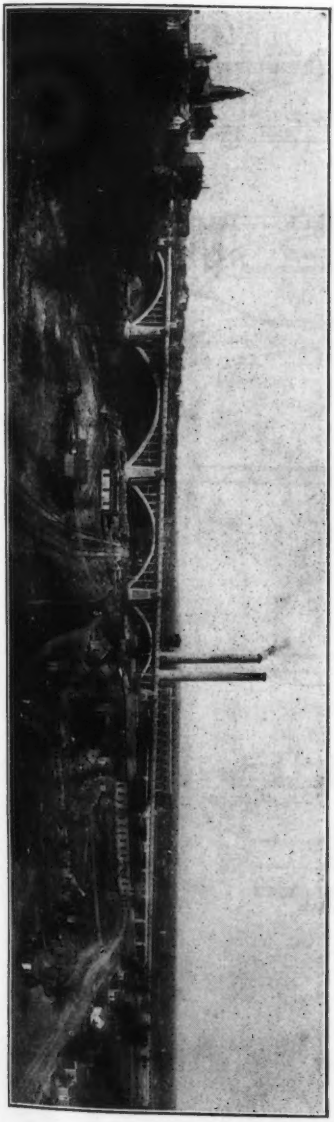

Packing Houses
North Fort Worth
Trinity River
West Fork
MAIN STREET
EAST FOURTH STREET Earth Fill VIADUCT Fill
Earth Fill VIADUCT
Fort Worth Proper
WEST SEVENTH STREET VIADUCT
FT. WORTH & RIO GRANDE RY.
ST. LOUIS SOUTHWESTERN RY.
GULF COLORADO & SANTA FE RY.
FORT WORTH & DENVER CITY RY.
C. Rl. & G. RY.
Waterworks
TOPOGRAPHICAL MAP
OF PART OF
FORT WORTH
IN THE VICINITY OF TRINITY
RIVER AND BRANCHES
SHOWING LOCATION OF LEVEE
AND VIADUCTS
SCALE OF FEET
0 500 1000 2000
N

FIG. 1.



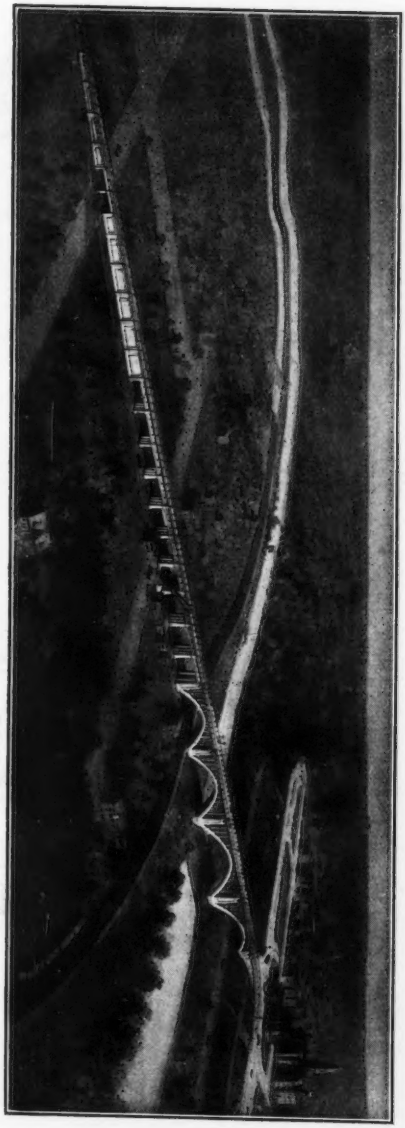
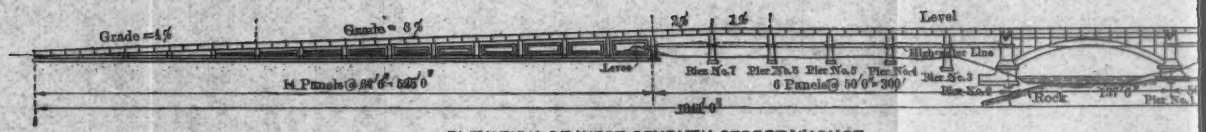
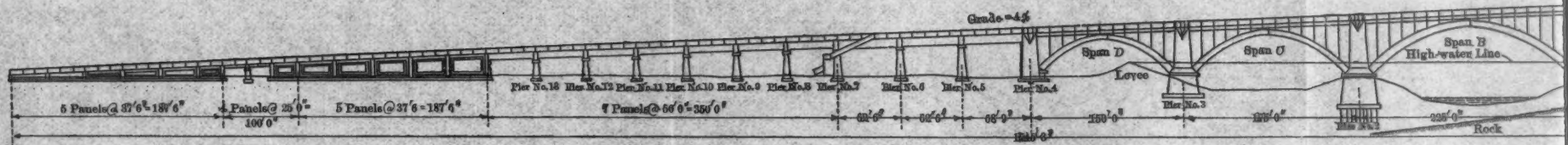


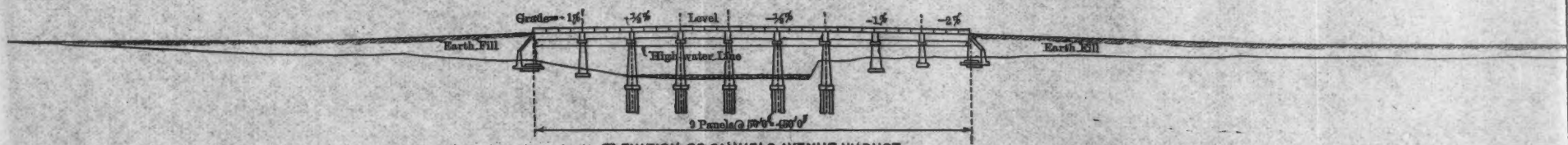
FIG. 2.—MAIN STREET VIADUCT, FORT WORTH, TEXAS, FROM AN ARTIST'S PERSPECTIVE.



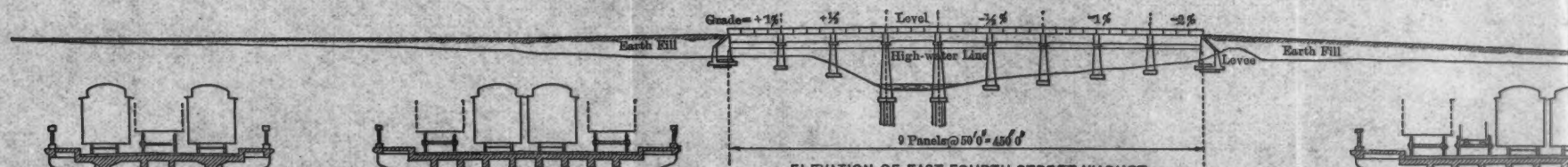
ELEVATION OF WEST SEVENTH STREET VIADUCT.



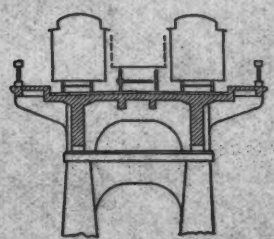
ELEVATION OF MAIN STREET VIADUCT.



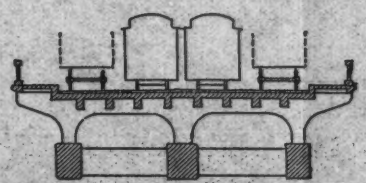
ELEVATION OF SAMUELS AVENUE VIADUCT.



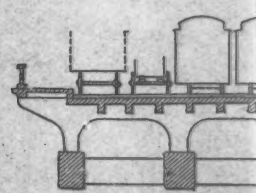
ELEVATION OF EAST FOURTH STREET VIADUCT.



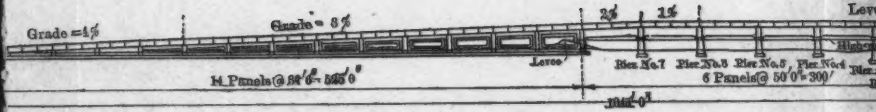
CROSS-SECTION THROUGH GIRDERS
SAMUELS AVE. AND EAST FOURTH ST. VIADUCTS.



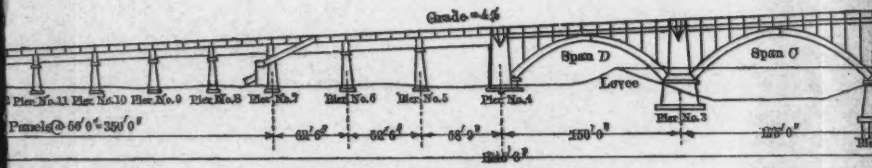
CROSS-SECTION THROUGH ARCH
WEST SEVENTH STREET VIADUCT.



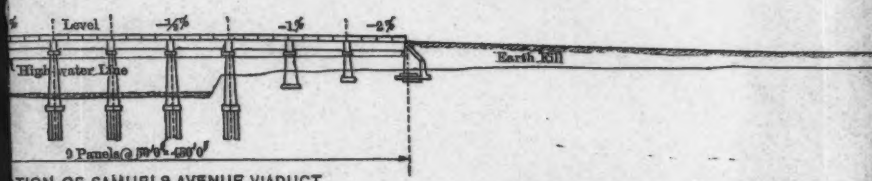
CROSS-SECTION TH
MAIN STREET V



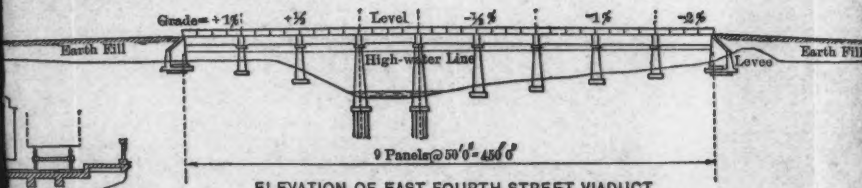
ELEVATION OF WEST SEVENTH STREET VIADUCT.



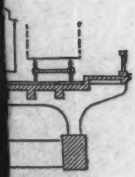
ELEVATION OF MAIN STREET VIADUCT.



TION OF SAMUELS AVENUE VIADUCT.

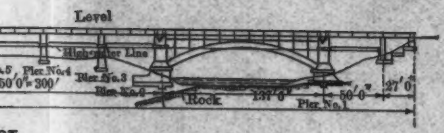


ELEVATION OF EAST FOURTH STREET VIADUCT.



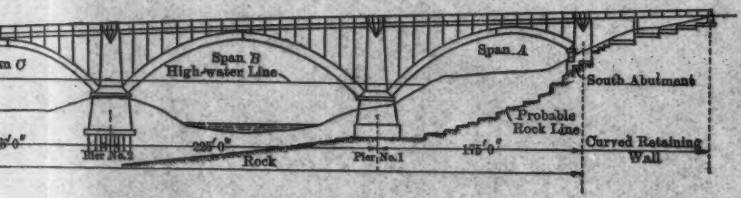
UGH ARCH
ET VIADUCT.

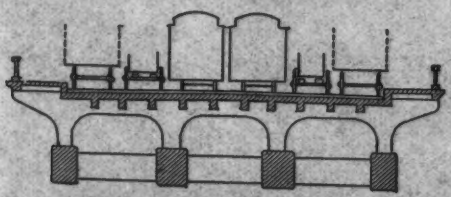
PLATE XV.
PAPERS, AM. SOC. C. E.
AUGUST, 1914.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



GENERAL ELEVATIONS
AND SECTIONS
FORT WORTH VIADUCTS
FORT WORTH, TEXAS

CT.





CROSS-SECTION THROUGH ARCH
MAIN STREET VIADUCT.

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walls. The other two structures each have nine 50-ft. girder spans, and are 450 ft. long, exclusive of the approaches, which consist of earth fills without retaining walls. All the spans are measured from center to center of piers.

In general, ornamentation is confined to the main lines of the structures, and to the use of paneling and mouldings. Railings are simple in outline, and designed with a view to easy construction. Balconies are used at the tops of the main piers at Main and West Seventh Streets, to finish them off properly. The lines of the structures are simple, and there is no applied ornamentation.

Foundation Conditions.—With the exception of a portion of the Main Street and West Seventh Street crossings, rock lies at too great a depth below the surface to be reached, except at excessive cost. At Main Street bed-rock lies at a depth of from 40 to 50 ft. below the ground surface north of the stream. On the south side the rock rises, and crops out at the surface on top of the bluff. Some twenty years ago a landslide occurred on the south bank of the stream. The earth and loose rock overlying bed-rock, slid toward the river, partly wrecking the old bridge. This slide extended for several hundred feet along the stream. It was probably caused by water seeping down from the bluff and softening the material in the hillside.

The material on the south side of the river consists of loam and clay interspersed with loose rock and boulders brought down by the slide. This material is underlaid with ledges of rock, varying in thickness from 6 in. up, with thin layers of hard clay or shale between. The layers are horizontal, and increase in thickness with the depth. The material on the north bank, to a depth of 10 or 12 ft., consists of a black, sticky, clay-like substance, which has the property of remaining very hard when moist and not exposed to the air, but which disintegrates when exposed to dry air for a few days. Under this surface material there is a very dense yellow clay, underlaid with sand. This sand increases in coarseness with the depth, and merges into gravel near bed-rock. The surface material was tested at a depth of 8 ft., and sustained a load of 3.3 tons per sq. ft. without appreciable settlement. The material north of the stream at Main Street is fairly typical of that at the other crossings.

River Conditions.—The Trinity River, though normally a small stream, hardly larger than a creek, is subject to sudden freshets, and

carries large quantities of drift. The banks are steep and the run-off is rapid. As an example of the rapidity of the rises, may be mentioned a flood that occurred during the summer of 1912, before work started, and another in September of 1913, while work was under way. During the first mentioned freshet a rise of 23 ft. occurred in 24 hours. This rise was gradual compared with the last mentioned flood, where the rise, measured at West Seventh Street, amounted to a total of 16 ft. in about 1½ hours. The first 14 ft. of this rise took place in about 1 hour. Such floods as these are not at all unusual, and make work in the bed of the stream decidedly hazardous.

Selection of Type of Structure.—Because of the conditions mentioned above, it was necessary to adopt a type of structure that would not be injured by such slight unequal settlement as, under the circumstances, might be expected to occur. It was also thought advisable to use, at least for the arch spans, a method of construction that would not require falsework in the stream.

After a careful consideration of various types, it was decided to use, for the main spans of the two larger viaducts, three-hinged, ribbed arches, with structural steel reinforcement designed to support the weight of the forms and plastic concrete of the ribs and braces during construction. For the approach spans and for the river spans of the smaller viaducts, girder spans were adopted.

The three-hinged arch was selected because it would not be strained by unequal settlement, because the stresses are statically determinate, and temperature stresses are eliminated. Ribbed construction was adopted as being light and best adapted to the use of hinges, and also because no water-proofing would be required. Structural reinforcement for the ribs and braces was used in order to dispense with falsework, as far as possible.

Preliminary Design.—Before beginning the actual designing, a considerable amount of preliminary work was done in order to determine the most economical type of structure for various heights, and also to determine the economical span lengths for each type. Three types were considered, as follows: earth fills enclosed by semi-gravity retaining walls, girder spans, and arch spans.

In this preliminary work sufficient designing and laying out was done to enable the various quantities to be determined with sufficient accuracy for the purpose in view. These quantities were multiplied

by estimated unit prices, and the cost per linear foot of structure, of each type, for various heights and span lengths, was obtained. These results were then plotted, and the economical span lengths were determined, as well as the points at which it was theoretically economical to change from one type of construction to another. The information obtained from the plots was followed as closely as circumstances would permit. In some cases, however, the physical conditions encountered determined both the span lengths and the type of structure. For example, at Main Street, arch spans were used over the levee, and between the south abutment and Pier No. 1. Girder spans would have been cheaper at these points, if the height only were considered. It was not permissible to place piers in the levee, and a long span had to be used. It was not thought advisable to divide the span between the South Abutment and Pier No. 1, on account of the possibility of a recurrence of the slides mentioned previously; in other words, the fewer piers on the south bank of the stream at this point, the better. Consequently, arch spans were used for these portions of the crossing.

Girder spans would have been cheaper than the arch at West Seventh Street, but, as this structure is near one of the city parks, and in a high-class residence district, an arch was used over the stream, for esthetic reasons.

Girder spans were used throughout for the Samuels Avenue and East Fourth Street crossings, for the sake of economy, and to afford the maximum clear opening with minimum height. This required the use of falsework in the stream. This was undesirable, but it was considered that as in this case the girder spans were considerably cheaper than self-supporting arches, and that as these crossings were situated so that the force of the current at flood stage was much less than at the other crossings, girder spans would answer the purpose.

The selection of the three-hinged ribbed arch with forms supported from the reinforcement, was governed principally by the character of the foundations, and by the erratic behavior of the stream. However, at Main Street, estimates were made of the cost of this construction as compared with that of a three-hinged ribbed arch supported on falsework in the usual manner. In making this comparison, the cost of the falsework and the rib reinforcement required for this scheme was compared with that of the rib reinforcement of the arches

as built. The hinges, and such portions of the rib forms as were common to both schemes, were not included in the estimate.

It was found that the structure on falsework would have cost slightly less than as built. This was caused by the fact that Spans *A*, *C*, and *D* are too low for economy with the self-supporting type of construction. Span *B*, however, is high enough to make this type economical. It was decided to use the self-supporting type for all spans, because of the danger from floods, the unstable character of the soil under Span *A*, and in order to avoid the troubles due to settlement of the falsework.

The selection of the self-supporting type was amply justified by events which will be mentioned later, under the head of "Construction."

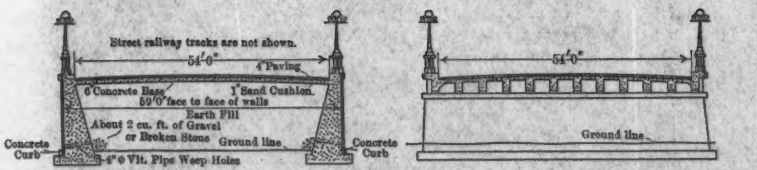
General Layouts.—After the preliminary designing had been finished, and the data obtained had been arranged for use, the work of preparing the general plans was started. The Main Street structure is typical of the work, and will be used as an example throughout the remainder of this paper.

The general plan of this crossing, Plate XVI, shows its location with reference to the principal streets and public buildings. It will be seen that a direct approach to the south end of the structure was prevented by the Court House and Jail buildings, which block Main Street at this point. It was desirable, therefore, to make the south approach a double one. One branch, carrying north-bound traffic, connects with Commerce Street, and the south-bound traffic is accommodated by the other branch, which connects with Houston Street. Sufficient property has been acquired to allow the street corners to be rounded off, so that the approaches may follow easy curves.

Curved retaining walls and railings are being constructed south of the south abutment, where the viaduct proper ends. It is expected that the block now occupied by the Jail and other buildings, will be cleared by the City, and will be parked to form a pleasing approach to the crossing.

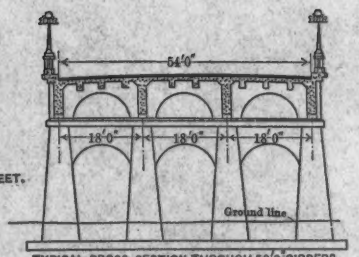
The general elevation was laid out to fit the conditions, keeping in mind the data obtained from the preliminary designs. It will be seen by referring to this general elevation, Plate XVII, that a symmetrical structure was not possible, owing to the contour of the ground.

Structural Features.—Before going into the matters of loading, unit stresses, and methods of design, it may be well to give a descrip-

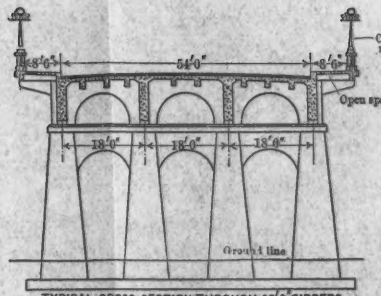


TYPICAL CROSS-SECTION THROUGH RETAINING WALLS.

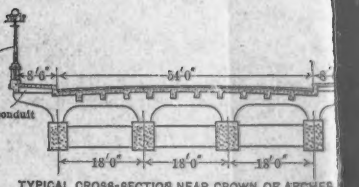
CROSS-SECTION THROUGH GIRDERS AT N. THIRD STREET.



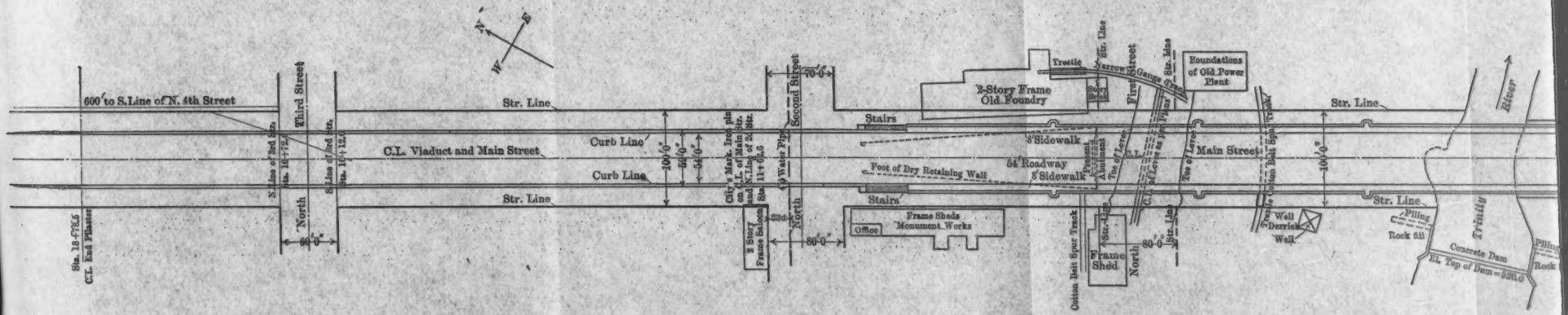
TYPICAL CROSS-SECTION THROUGH 50'0" GIRDERS.

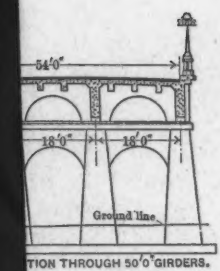


TYPICAL CROSS-SECTION THROUGH 62'6" GIRDERS.

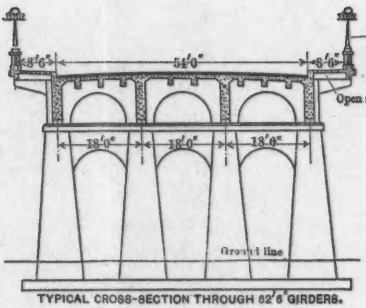


TYPICAL CROSS-SECTION NEAR CROWN OF ARCHES.

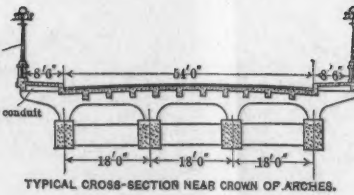




TION THROUGH 50'0" GIRDERS.



TYPICAL CROSS-SECTION THROUGH 62'6" GIRDERS.



TYPICAL CROSS-SECTION NEAR CROWN OF ARCHES.

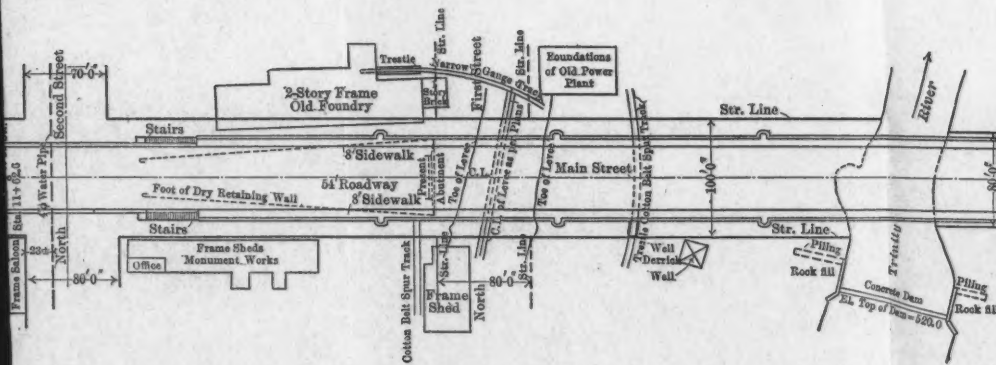
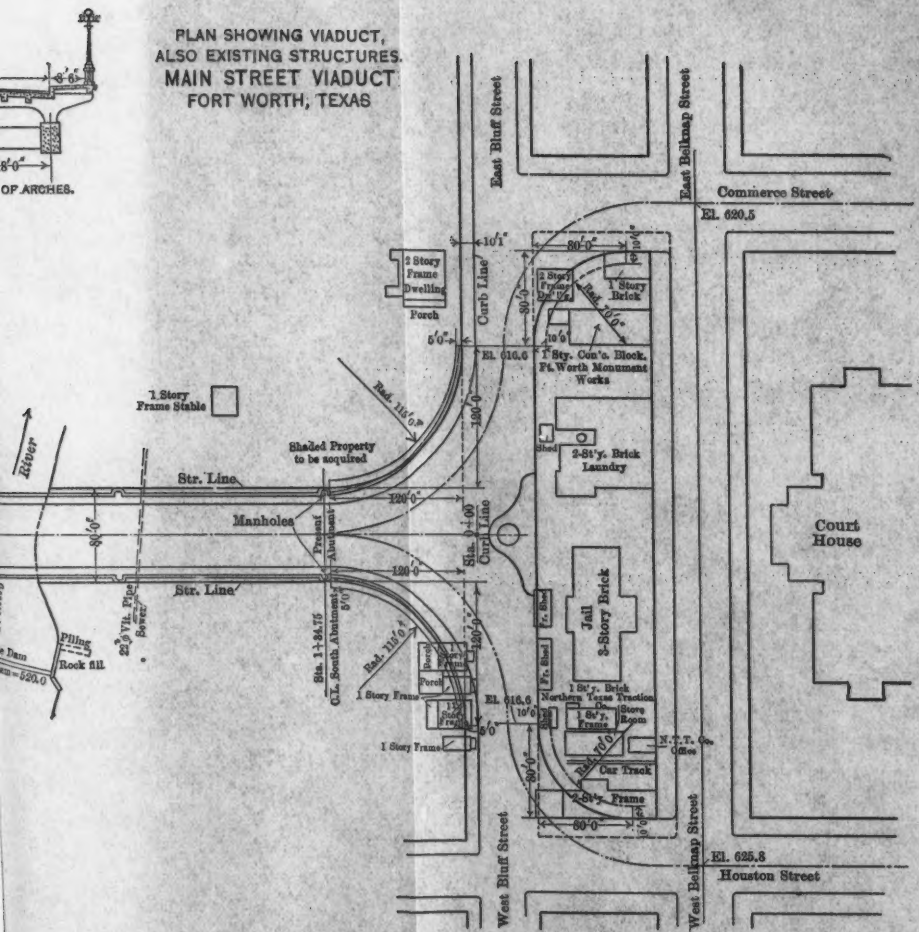


PLATE XVI.
PAPERS, AM. SOC. C. E.
AUGUST, 1914.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.

PLAN SHOWING VIADUCT,
ALSO EXISTING STRUCTURES.
MAIN STREET VIADUCT
FORT WORTH, TEXAS



tion of some of the features of the Main Street Viaduct, in order to show the character of the construction, and to make clearer some of the problems involved in the design.

As shown by the plans, the deck consists of slabs carried on longitudinal stringers, which connect to floor-beams. These floor-beams are in turn carried by the four main longitudinal girders of the girder spans, or by the spandrel posts, which rest on the four ribs of the arch spans. The sidewalks are carried on cantilever extensions of the floor-beams outside of the outer girders or ribs. It will be noticed that, for the girder spans, such stringers as would come close to the girders are omitted, and their place is supplied by the cantilevered top flanges of the main girders. Those cantilevered top flanges also provide the necessary compressive area for the girders.

Two ribs or main girders would have been more desirable in some respects than four, as this arrangement would have done away with the continuous girder action of the floor-beams, and made the reactions on the girders and ribs statically determinate. The heavy concentrated loading and the width of the roadway, however, would have made the floor system extremely heavy. Therefore the four-rib arrangement was adopted for this crossing. At West Seventh Street, three ribs were used, and two lines of main girders for the other two structures.

The same length of floor system panels was adopted for all the structures. This made the design of the stringers practically uniform throughout. The length used was 12 ft. 6 in. from center to center of floor-beams, which gave stringers of reasonable size and brought the loads on the girders and arch ribs at reasonably short intervals. The floor-beams are of uniform thickness throughout, which thickness, 17 in., is the same as that of the spandrel posts. The posts are purposely made thinnest in the direction of the axis of the structure, so that the bending caused by the changes in the length of the deck due to temperature, will not over-stress them.

Expansion joints are provided in the girder spans at each pier, and in the deck of the arch spans, at the crown, and at each pier. The main girders slide on top of the small piers, and the stringers slide in recesses or pockets in the floor-beams. All expansion joints are packed with tar-paper and asphalt to form a closed but compressible joint, and all main girder joints are masked by pilasters. Planed steel, bed-

and bearing-plates are used under the expansion end of each girder and stringer. These plates have anchorage angles riveted to them, and are further anchored with bent anchor-bolts passing through holes in these angles. The sliding members, and those on which the sliding members rest, are also reinforced at these bearing points with loops of steel, to provide for the shearing stresses which result from the friction of the plates on each other. In order to reduce this friction as much as practicable, the bed- and bearing-plates are planed in the direction of movement, and the sliding surfaces in contact are heavily coated with tallow. U-shaped copper plates, filled with asphalt, are used in the roadway slabs to close the opening. Similar plates placed in the reversed position to those in the roadway slabs, and with the asphalt filling omitted, are used in the sidewalk slabs and curbs. These copper plates are used at each expansion joint in the structure.

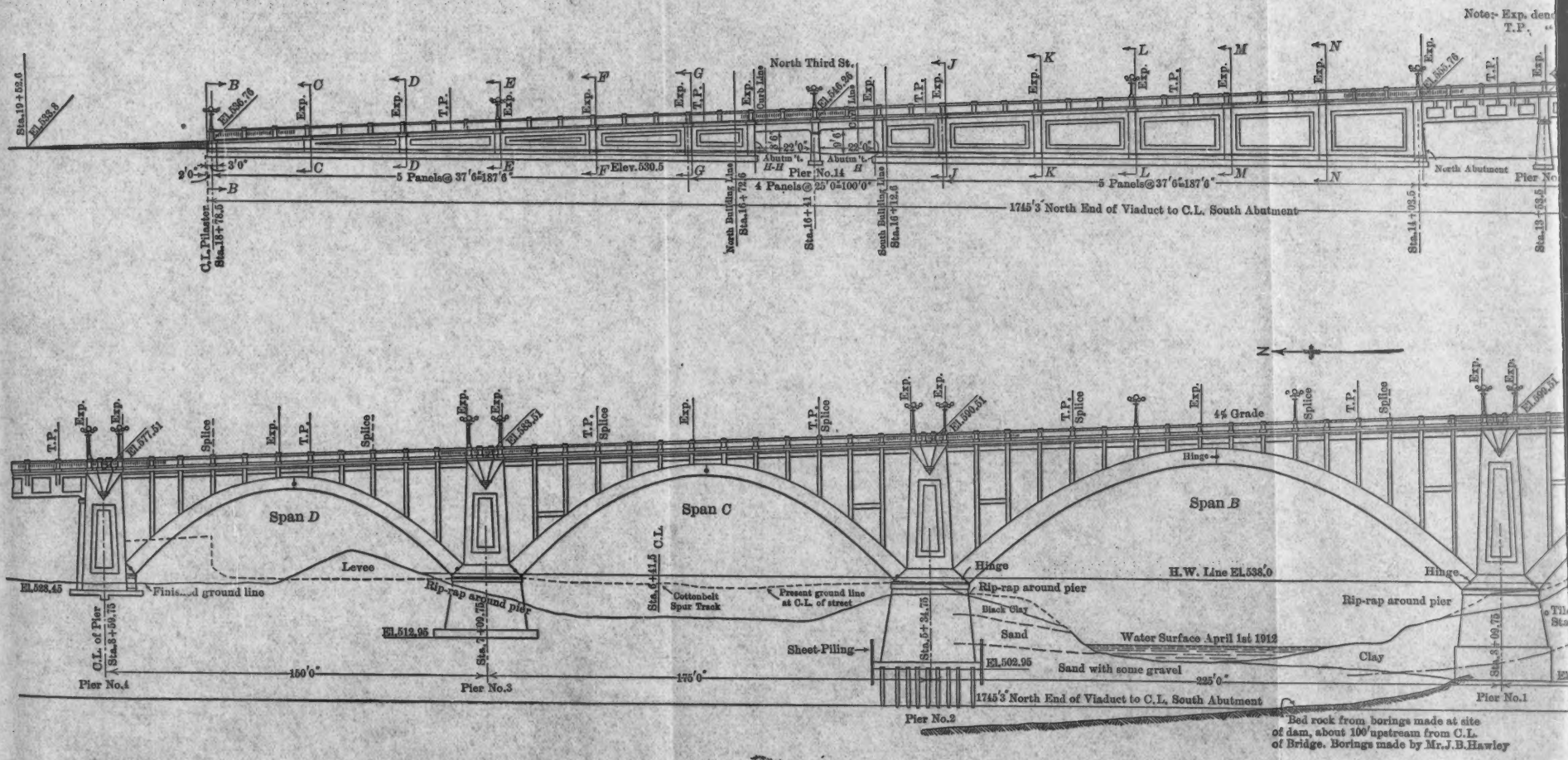
The arch ribs are braced together at the foot of each spandrel post by transverse braces, which are of the same width as the spandrel posts and floor-beams, and as deep as the ribs will allow. These braces are placed in vertical planes.

The arch ribs are of uniform thickness, but vary in depth, decreasing regularly from the haunch toward the crown. This is contrary to the theoretical shape of a three-hinged rib, which should be deepest at about the quarter point of the span, and diminish in depth toward the haunch and crown. This departure from the theoretical shape was made for the sake of appearance only. As the hinges are concealed in the finished rib, it was considered that the ribs would look best if made to follow the lines of a hingeless rib.

Each hinge consists of two steel castings, having ball and socket joints. At the haunches one of these castings is built into the pier, and the other is bolted to the structural rib reinforcement, as are also the castings forming the crown hinge.

All the piers for the arch spans are of considerable width in order to resist overturning forces and to give the appearance of massiveness required by the long spans. They are hollowed out as much as is consistent with strength and economy of forms, in order to reduce their weight and to save material.

The small piers supporting the girder spans consist of four square shafts, battered on the four faces. These shafts rest on a common base and are connected at the top by a cap and diaphragm of T-section.


North Third St.
Pier No.14
Abutm't. H-H
Abutm't. H
22'0"
9'6"
5 Panels @ 37'6"=187'6"
4 Panels @ 25'0"=100'0"
1745'3" North End of Viaduct to C.L. South Abutment
North Abutment
Pier No
Elev. 530.5
C.L. Pilaster
North Building Line
South Building Line
Curb Line
T.P.
Exp.
Splice
Span D
Span C
Span B
Levee
Finished ground line
Rip-rap around pier
Cottonbelt Spur Track
Present ground line at C.L. of street
4% Grade
Hinge
H.W. Line El.538.0
Black Clay
Sand
Water Surface April 1st 1912
Clay
Sand with some gravel
Sheet-Piling
El.528.45
El.512.95
El.502.95
C.L.
C.L. of Pier
150'0"
175'0"
225'0"
Pier No.4
Pier No.3
Pier No.2
Pier No.1
Bed rock from borings made at site of dam, about 100' upstream from C.L. of Bridge. Borings made by Mr. J.B.Hawley

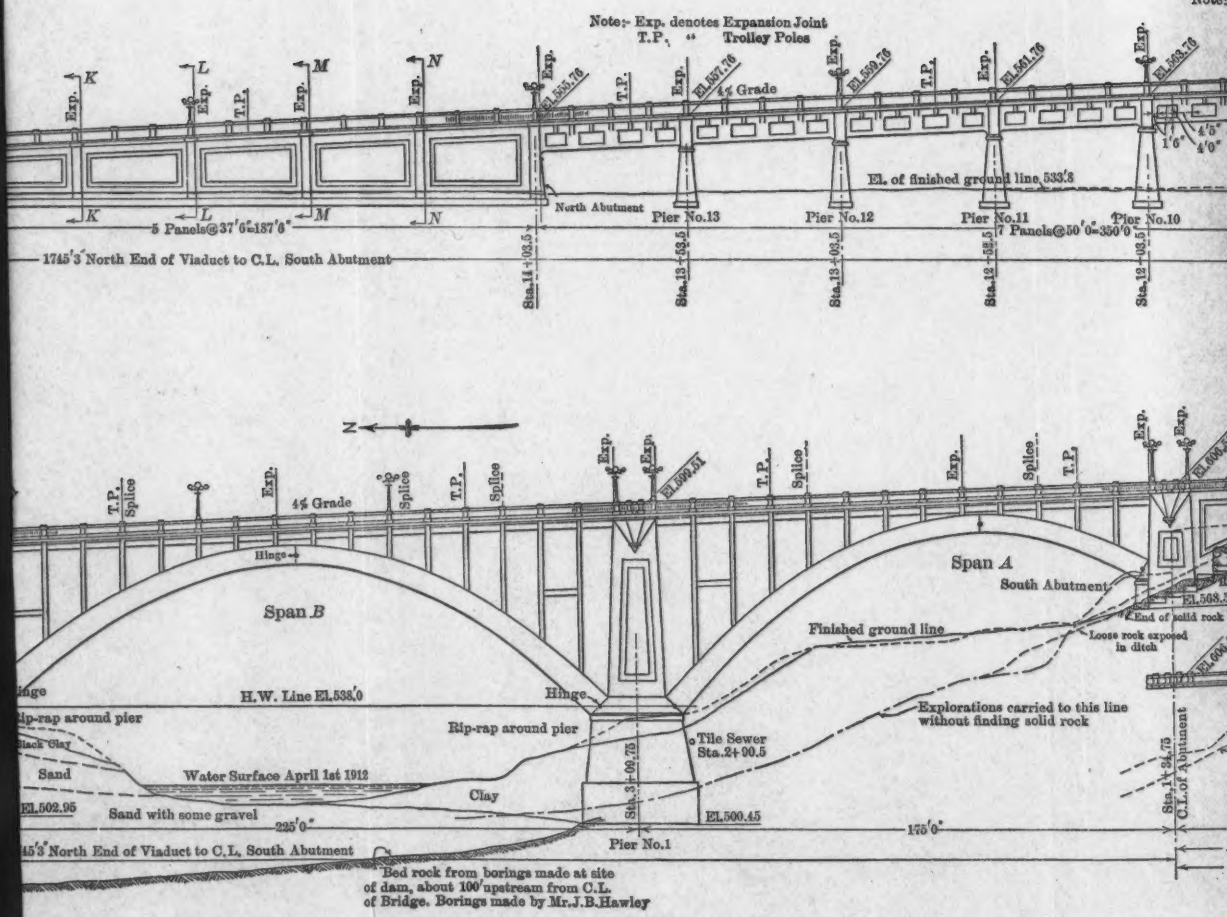
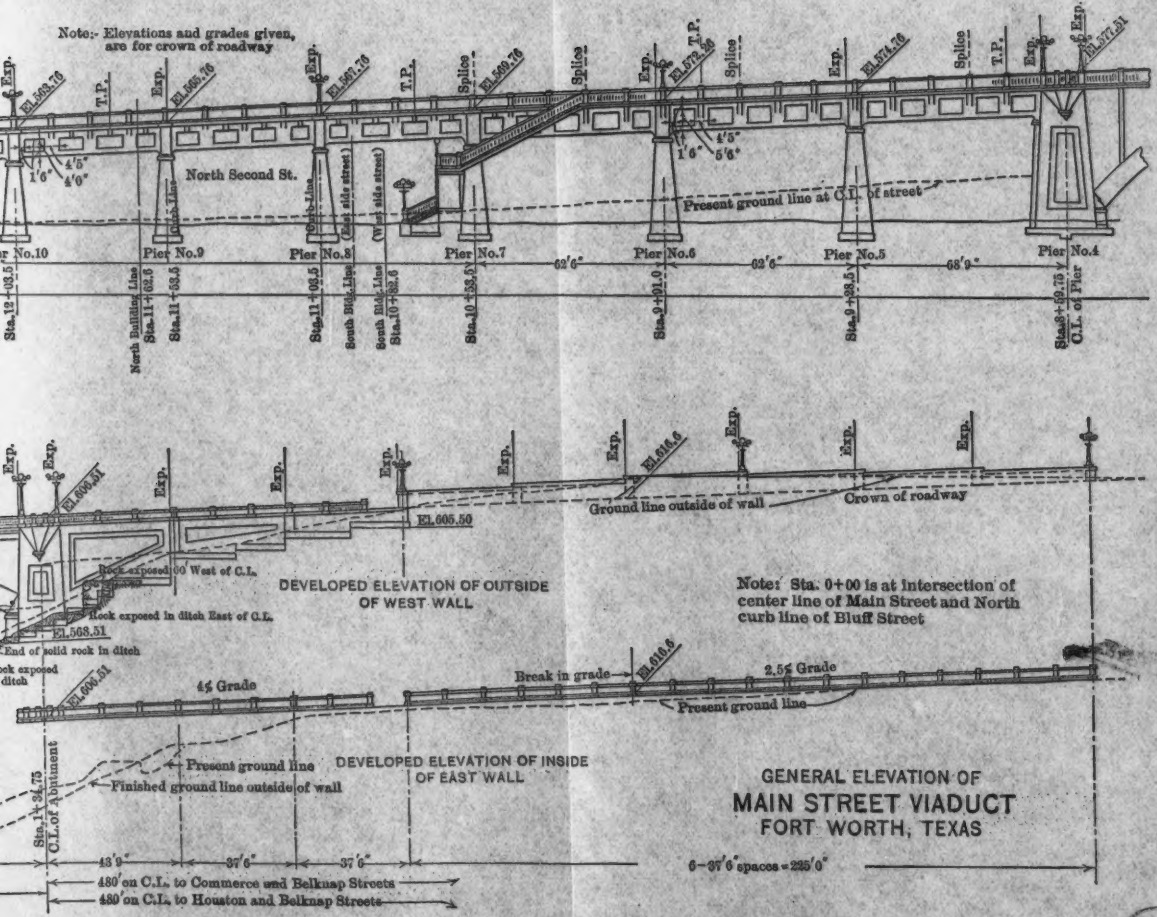

Note:- Exp. denotes Expansion Joint
T.P. " Trolley Poles
K
L
M
N
Exp.
T.P.
EL.555.76
EL.557.76
EL.559.76
EL.561.76
EL.563.76
4% Grade
4'5"
4'0"
1'6"
El. of finished ground line 533'.8
North Abutment
Pier No.13
Pier No.12
Pier No.11
Pier No.10
5 Panels@37'6"=187'6"
7 Panels@50'0"=350'0
1745'3" North End of Viaduct to C.L. South Abutment
Sta.14+03.5
Sta.13+53.5
Sta.13+03.5
Sta.12+53.5
Sta.12+03.5
Z
T.P.
Splice
Exp.
4% Grade
EL.600.51
Hinge
Span B
Span A
South Abutment
End of solid rock
Loose rock exposed in ditch
Finished ground line
H.W. Line El.538.0
Rip-rap around pier
Explorations carried to this line without finding solid rock
Tile Sewer Sta.2+90.5
Black Clay
Sand
Water Surface April 1st 1912
Clay
Sand with some gravel
EL.502.95
EL.500.45
225'0"
175'0"
Sta.3+09.75
Sta.1+34.75
C.L. of Abutment
45'3" North End of Viaduct to C.L. South Abutment
Pier No.1
Bed rock from borings made at site of dam, about 100' upstream from C.L. of Bridge. Borings made by Mr. J.B. Hawley

PLATE XVII.
PAPERS, AM. SOC. C. E.
AUGUST, 1914.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



GENERAL ELEVATION OF
MAIN STREET VIADUCT
FORT WORTH, TEXAS

UNIV. OF MICH.

The retaining walls are of the semi-gravity type. The toe of the section is reinforced for bending stresses, and the reinforcement extends high enough up the back of the wall to relieve the concrete of undue tensile stresses. Expansion joints are provided at intervals of 37 ft. 6 in., and pilasters are used at these points to mask the joints. The walls have a small coping and are paneled to relieve the appearance of flatness.

The South Abutment is founded on rock, as is also Pier No. 1, which is on the south bank of the stream. Pier No. 2, on the north bank, is supported on a spread foundation, with a portion of the load carried on timber piles. All other piers and walls are carried on spread footings, resting on the strata previously referred to, at depths ranging from 4 to 18 ft. below the finished ground surface.

Railings, and the stairways leading down to the street level at North Second Street, are of reinforced concrete, cast in place, except the railing panels and hand-rails, which were cast in moulds and erected.

The stairways, with the exception of the steps leading to the lower landing, are supported entirely from the piers and girders by structural steel cantilever beams encased in concrete. This method was adopted, instead of supporting partly on the piers and girders and partly on independent posts and foundations, in order to avoid the danger of damage due to unequal settlement. The lower steps were built on independent foundations, with a sliding joint between the steps and the remainder of the stairway, to take care of unequal settlement.

Provision is made for carrying electric wires and small pipes under the sidewalks, with ducts leading into the railing posts and up to the lighting fixtures.

In order to reduce the dead weight of the deck as much as possible, the usual ballasted track construction was dispensed with. Anchor-bolts were set into the slab over the stringers, and the rails were secured to these bolts with cast-steel clips. Steel bearing-plates were provided under the rails at each pair of bolts, which occurred at intervals of 27 in. The rails were lined and brought to proper level, with eccentric washers and shim plates. A fiber pad, ½ in. thick, was placed between the rail and the bearing-plate, as a shock absorber. This method of securing the rails is indicated on the plans.

The reinforcement of the arch ribs and braces consists of medium-steel structural shapes, supplemented by reinforcing bars. Structural

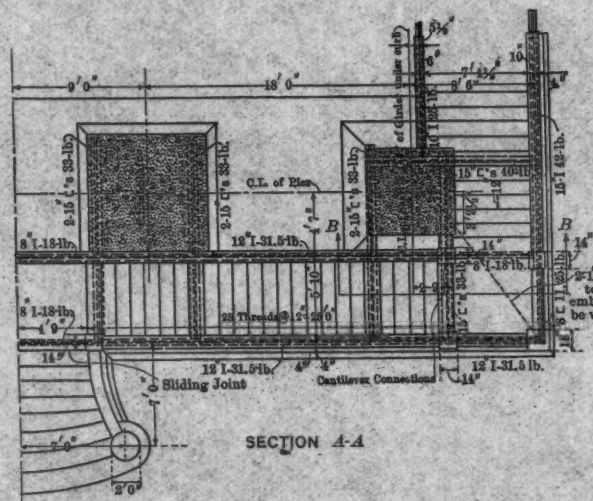
shapes are also used for the main supports of the stairways. All other reinforcement consists of square twisted bars, of high elastic limit. These were substituted for the round, mechanical-bond bars called for on the plans, and were made equivalent in cross-section to the round bars which they replaced. All bars, except some of the large sizes, were rolled from the heads of rails, and were hot-twisted. Cold-twisted bars, from medium-steel billet stock, were used for the large sizes, on account of the difficulty of getting rail heads of sufficient volume for this purpose. Tests of the two classes of material showed that they were practically equal in quality.

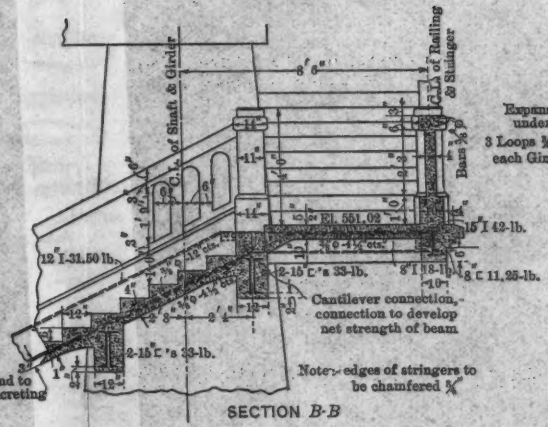
As shown on the plans, the rib reinforcement consists of curved lattice girders, to the ends of which are bolted the cast-steel hinges. The chords of these girders are made up of four angles arranged in the shape of a cross. All web members are made up of two angles. Three of these steel girders are used in each inner, and two in each outer, rib of each span. The individual steel girders are laced together in the plane of the top and bottom chords, and are also connected by transverse frames, in vertical planes, under each post. The transverse braces connecting the four ribs are reinforced by lattice girders which connect to the rib reinforcement. No reaming or painting is called for. The hinge castings are bolted to the steel ribs, and all other connections, both shop and field, are riveted.

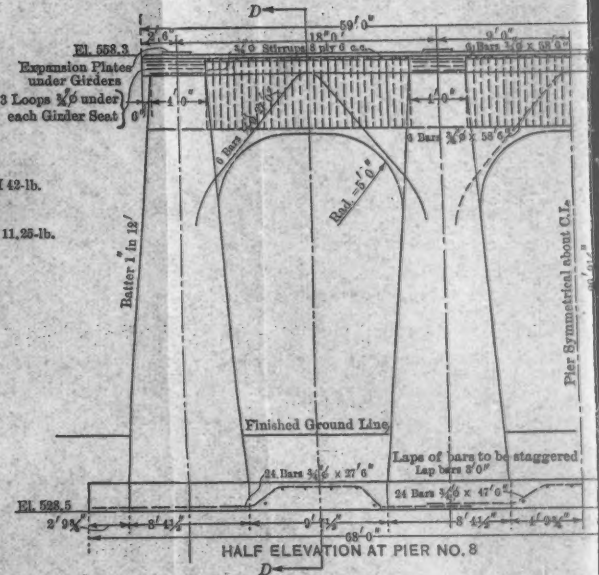
At each upper panel point of the steel ribs, clip angles were provided, to which were secured the transverse timber beams, from which the rib forms were supported. The method of supporting these forms is indicated on the plan showing the arch rib reinforcement.

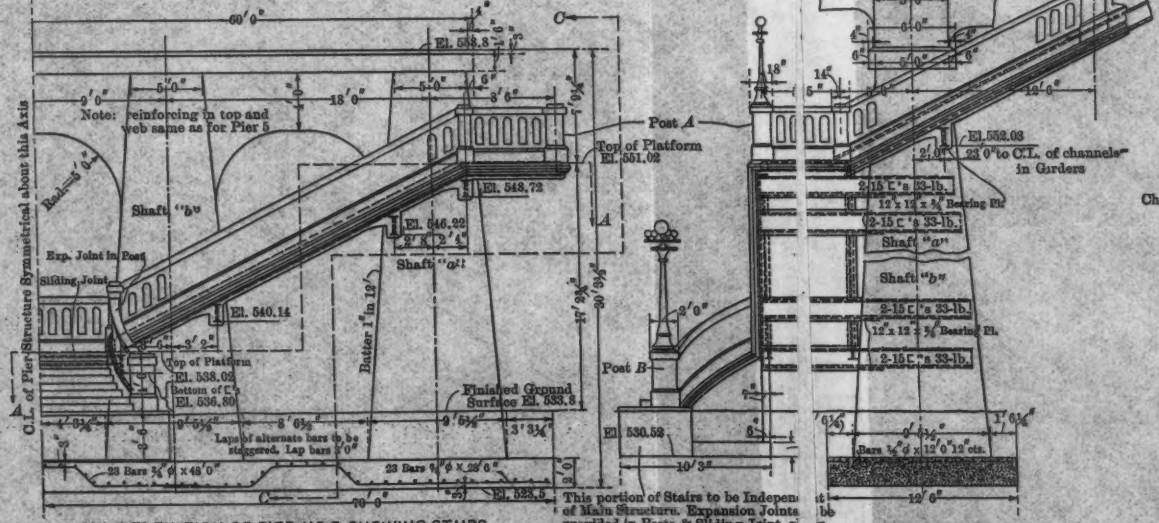
A system of lateral rods, for use during erection, was provided in the plane of each chord of the ribs. These rods were used to line up the steelwork accurately, and to hold it in line while the ribs were being concreted. The rods were not removed until after the completion of the work. The transverse braces, mentioned previously, are also reinforced in each face with continuous bars, the function of which is to resist bending in the braces due to transverse forces.

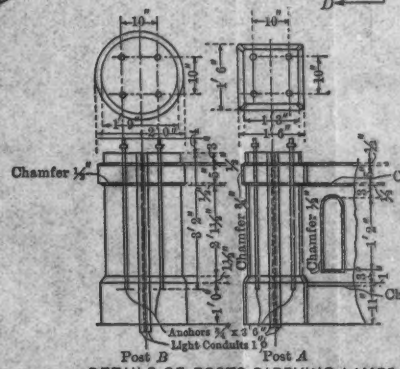
In addition to the steel reinforcement required to resist the calculated stresses in the members, a certain quantity of steel is used to prevent the formation of shrinkage, settlement, and temperature cracks. This steel is used in the exterior faces of thin walls, in a longitudinal direction in the slabs, and to reinforce the corners where



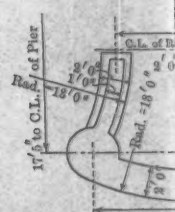






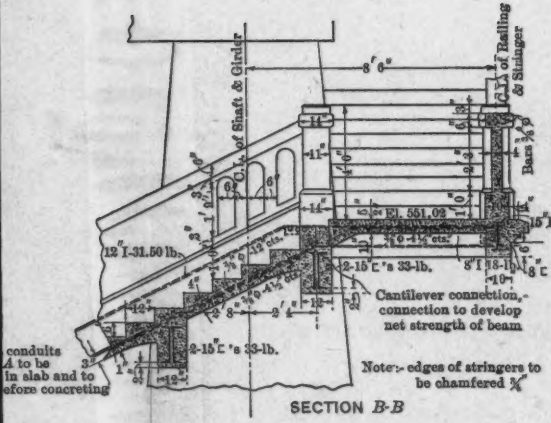


DETAILS OF PIERS 7 AND 8
MAIN STREET VIADUCT
FORT WORTH, TEXAS

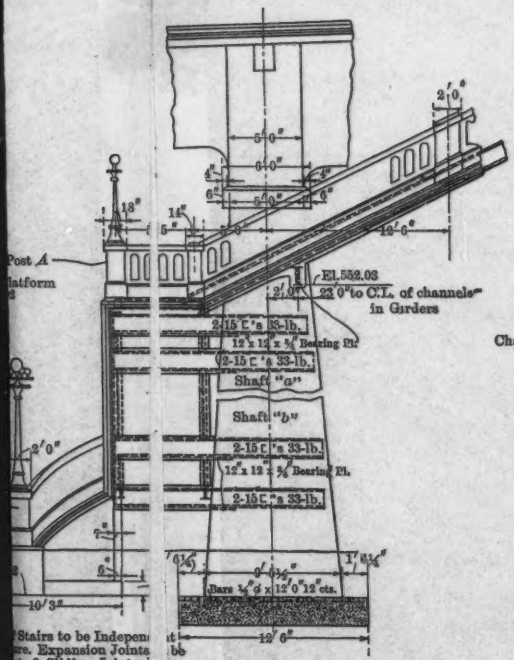


GENERA
Reinforcing bars to be secu
at all laps.
All vertical edges of piers
All other edges & squ.res
All reinforcing bars to have
bond; to be of sectional Are
and to have elastic limit not
Open-Hearth Structural St
Use ⅞" rivets where possibl
to be 12000 lb. per sq. in. for
for bearing
Materials and workmanshi
Manufacturers Standard Spe

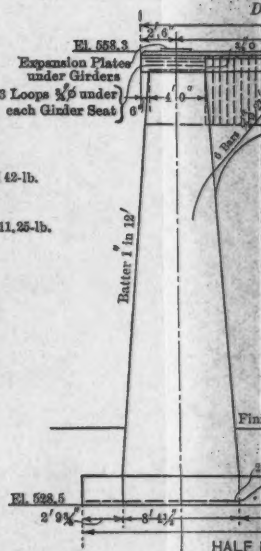
MIX OF
Stairs and Railing
1-part Portland cement
2- " sand
4- " broken stone
All by

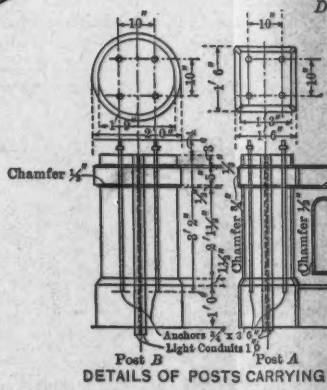


SECTION *B-B*



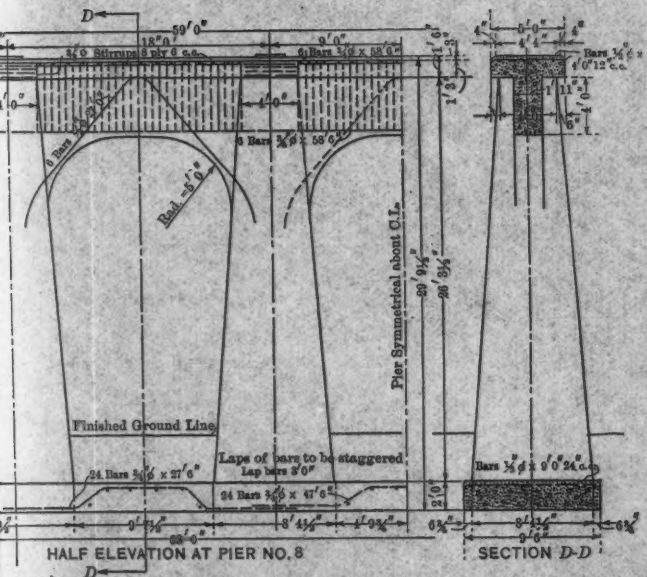
SECTION *C-C*

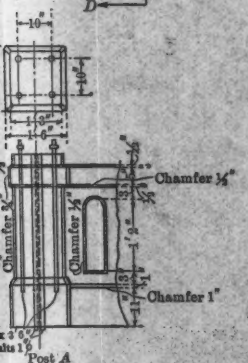




DETAILS OF POSTS CARRYING

DETAILS OF PIERS 7 AND 8
MAIN STREET VIADUCT
FORT WORTH, TEXAS





TS CARRYING LAMPS

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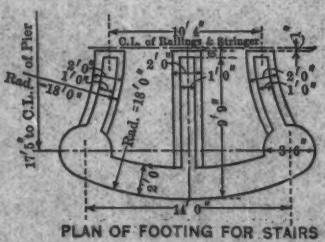


PLAN OF FOOTING FOR STAIRS

GENERAL NOTES

Reinforcing bars to be securely wired together at all laps.

All vertical edges of piers to be chamfered thus;

All other edges & square corners of piers " " "

All reinforcing bars to have some form of Mechanical bond; to be of sectional Area equivalent to that shown, and to have elastic limit not less than 50000 lb. per sq.in.

Open-Hearth Structural Steel to be used.

Use ⅞" rivets where possible. Allowable stresses on rivets to be 12000 lb. per sq. in. for shear and 24000 lb. per sq. in. for bearing

Materials and workmanship to be in accordance with Manufacturers Standard Specifications

MIX OF CONCRETE

Stairs and Railing	Piers
1-part Portland cement	1-part Portland cement
2- " sand	2½- " sand
4- " broken stone	5- " broken stone

All by volume

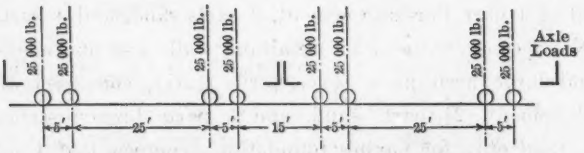
UNIV OF

two walls join. All braces, ribs, and posts are wrapped with heavy wire closely spaced, to which the longitudinal steel is secured.

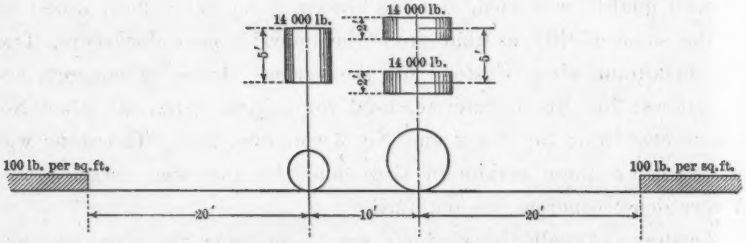
The concrete used in the construction of the viaducts is of three classes: No. 1 concrete, used for floor systems, girders, arch ribs, braces, spandrel posts, railings, stairs, and rib seats of large piers, composed of 1 part Portland cement, 2 parts sand, and 4 parts broken stone; No. 2 concrete, used for retaining walls and abutments, girder piers, and large arch piers (except rib seats), composed of 1 part Portland cement, 2½ parts sand, and 5 parts broken stone; No. 3 concrete, used only for paving foundations, composed of 1 part Portland cement, 3 parts sand, and 6 parts broken stone. A local sand of good quality was used, and the broken stone was a hard limestone of the same quality as that from the quarries near Jacksboro, Tex. The maximum sizes of stone for the various classes of concrete are as follows: for No. 1 concrete, used for railing, ¾-in., all other No. 1 concrete, 1-in.; for No. 2 and No. 3 concrete, 2-in. The stone was graded below these maximum sizes, and the dust was screened out. A very dense concrete was obtained.

Loading.—As all the viaducts are in or near the city, and are likely to carry the same class of traffic eventually, the same live loading was used throughout. The loading data are shown by Figs. 4 to 7, inclusive. The 50-ton electric cars and the 21-ton road roller shown thereon, were used for the members of the floor system, together with a uniform load of 100 lb. per sq. ft. on the sidewalks and remaining roadway surface. For maximum stresses in the floor-beams, the sidewalks were assumed to be unloaded. The arch ribs and the main longitudinal girders were designed for the foregoing car loading, together with 100 lb. per sq. ft. on the sidewalks and the remainder of the roadway. The road roller was also considered in connection with the car loading, with the foregoing uniform live load on the sidewalks and the remainder of the roadway. A load of 200 lb. per lin. ft. of each sidewalk was also used, in order to provide for the weight of pipes and conduits that might be carried in the spaces provided for them under the sidewalks. Pier footings were designed for a uniform load per linear foot of each car track, as given on Fig. 5, together with a uniform load of 100 lb. per sq. ft. on the sidewalks and the remaining roadway surface.

Impact was allowed for according to the following formula: $I = \frac{100\,S}{L + 300}$, where I is the impact to be added to the live load stress, S is the calculated maximum live load stress, and L is the loaded length, in feet, that produces the maximum stress in the member.



Two 50-Ton Electric Cars as above on each Track
Area covered by 4 cars as above 20'x 100'



One 21-Ton Road Roller as above on any part of Roadway
Width covered by Roller 10'

The car and roller given above, together with 100 lb. per sq. ft. on sidewalks and remainder of roadway, to be used in design of floor system, main longitudinal girders, and arch ribs. For Piers and footings, use 100 lb. per sq. ft. of sidewalks and Roadway and uniform track loading as per diagram, except Case II in design of arch Piers, where wheel loads are to be used to give maximum thrusts.

IMPACT

On sidewalks	none
Road Roller	25%
Other roadway loads and track loads	$I = S\frac{100}{L+300}$

where I = Impact to be added to the Live Load Stress.
S = Calculated maximum Live Load Stress.
L = Loaded Length, in feet, which produces maximum stress in the member.
No impact to be used in design of Piers and footings.

FIG. 4.

This impact allowance is one-third of that required by the original formula, submitted by the American Railway Engineering and Maintenance of Way Association, for impact on steel railway bridges, and was arbitrarily assumed. This formula was used only for car and

uniform roadway loads. Stresses produced by the road roller, which is a slowly moving load, were increased by 25% in all cases. No addition was made for impact to the stresses produced by the uniform sidewalk loading, as it was considered that when this sidewalk load was a maximum, that is, due to a closely packed crowd of people, it neces-

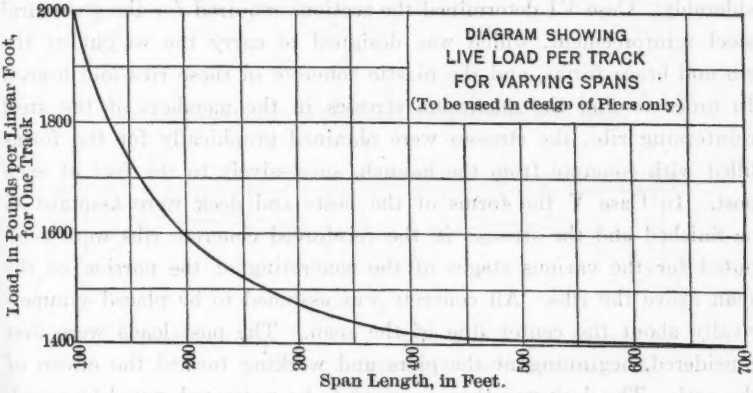
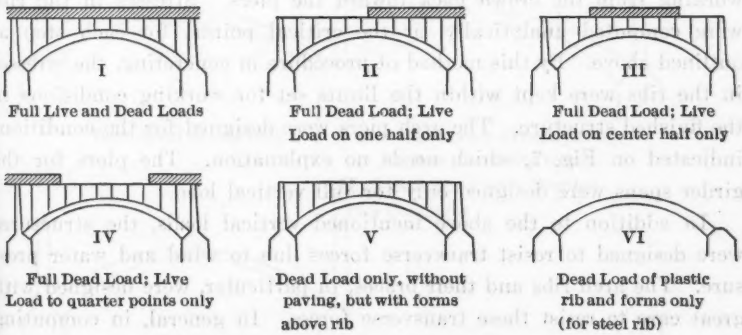


FIG. 5.



LOADING TO BE USED IN DESIGN OF ARCH RIBS

Note: In all cases where Live Load is called for, Wheel Concentrations shall be used and Allowance included for Impact

FIG. 6.

sarily would be nearly quiescent. The live loads for the piers were not increased for impact, as it was believed that the impact would be absorbed before reaching the footings.

The various conditions of loading for which the arch ribs were designed, are indicated on Fig. 6. Cases I to IV require no explana-

tion, except to say that they served to produce maximum stresses in the ribs of all spans except those of the unsymmetrical arch, Span *A*. On account of this lack of symmetry, these cases were elaborated on, in order to be sure that the maximum stresses had been found. Cases V and VI are construction conditions, and were elaborated on considerably. Case VI determined the sections required for the structural steel reinforcement, which was designed to carry the weight of the rib and brace forms, and the plastic concrete in these ribs and braces. In order to find the maximum stresses in the members of the steel reinforcing ribs, the stresses were obtained graphically for the forms filled with concrete from the haunch, successively to the foot of each post. In Case V the forms of the posts and deck were assumed to be finished and the stresses in the reinforced concrete ribs were computed for the various stages of the concreting of the portion of the span above the ribs. All concrete was assumed to be placed symmetrically about the center line of the span. The post loads were first considered, beginning at the piers and working toward the crown of the arch. The deck was then assumed to be concreted, panel by panel, working from the crown back toward the piers. Stresses in the ribs were computed analytically, at the critical points, for each step as outlined above. By this method of procedure in concreting, the stresses in the ribs were kept within the limits set for working conditions in the finished structure. The arch piers were designed for the conditions indicated on Fig. 7, which needs no explanation. The piers for the girder spans were designed only for full vertical load.

In addition to the above mentioned vertical loads, the structures were designed to resist transverse forces due to wind and water pressure. The arch ribs and their braces, in particular, were designed with great care to resist these transverse forces. In general, in computing these transverse forces, the water line was assumed to be at the top of the levee. Below this line the pressure due to the current was taken at 200 lb. per sq. ft. of vertical projection of each arch rib and spandrel post. Above this line the wind pressure was assumed at 30 lb. per sq. ft. of vertical projection of the structure, up to a line 10 ft. above the top of the roadway. The force assumed for the current corresponds, according to the formula used, to a velocity of about 8 or 9 miles per hour, which is probably not far from the actual velocity at flood stage.

At the West Seventh Street crossing, where the rise of the arch is small, and the high-water line comes well up on the arch rib, and where considerable drift is carried by the stream, more severe conditions

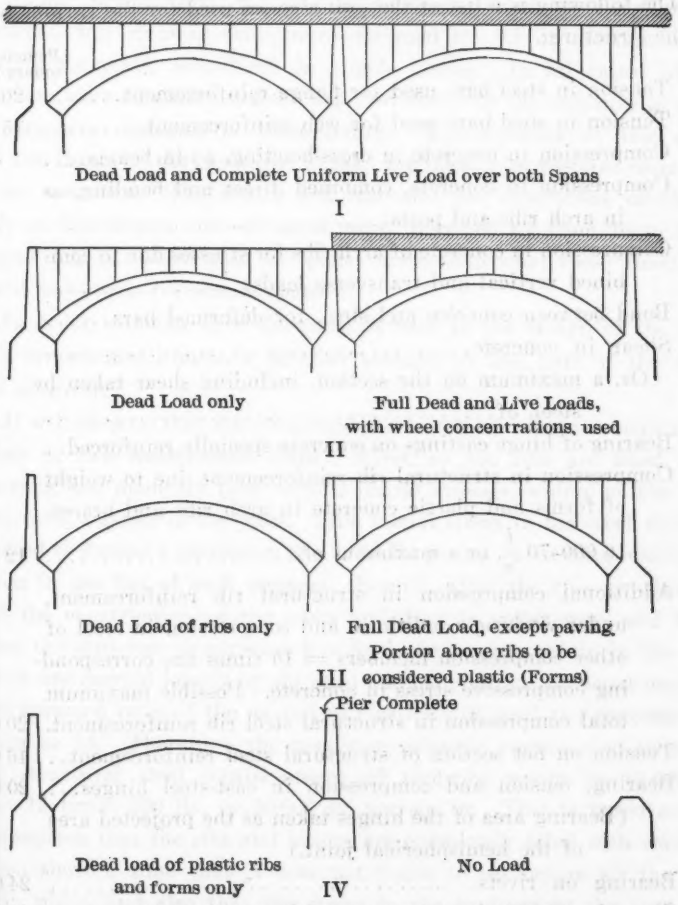


FIG. 7.

of transverse loading than those mentioned above, were assumed. In this case the water line was taken at the top of the arch rib, at the crown, in order to allow for the possibility of drift collecting against the span.

Specifications.—With certain modifications, in regard to unit stresses, "Watson's Specifications for Reinforced Concrete Bridges" were used.

The following is a list of the unit stresses used for the various parts of the structure:

	Pounds per square inch.
Tension in steel bars used for flange reinforcement	20 000
Tension in steel bars used for web reinforcement	15 000
Compression in concrete in cross-bending, as in beams	750
Compression in concrete, combined direct and bending, as in arch ribs and posts	500
Compression in concrete of arch ribs for stresses due to combined vertical and transverse loads	650
Bond between concrete and steel, for deformed bars	120
Shear in concrete	60
Or, a maximum on the section, including shear taken by steel, of	150
Bearing of hinge castings on concrete specially reinforced	800
Compression in structural rib reinforcement due to weight of forms and plastic concrete in arch ribs and braces, $16\,000-70\,\frac{l}{r}$, or a maximum of	12 500
Additional compression in structural rib reinforcement, under working conditions, and compression in steel of other compression members = 15 times the corresponding compressive stress in concrete. Possible maximum total compression in structural steel rib reinforcement.	20 000
Tension on net section of structural steel reinforcement	16 000
Bearing, tension and compression in cast-steel hinges	20 000
(Bearing area of the hinges taken as the projected area of the hemispherical joint.)	
Bearing on rivets	24 000
Shear in cast steel	10 000
Shear on rivets	12 000
	Pounds per square foot.
Bearing on soil at depth of 6 ft. below finished surface	5 000
For each 1 ft. increase in depth, more than 6 ft., add	100
	Pounds.
Load on each timber pile	20 000

Methods of Design.—There is nothing unusual in the design of the structures, with the exception of the methods of designing the arch ribs and braces. Briefly, sidewalk slabs and main girders were designed as simple beams, and continuous members, such as floor-beams, stringers, and roadway slabs, were designed for 0.8 of the maximum positive moments, considered as simple beams. In all cases, where members were continuous, steel was provided over the supports to take the negative moments at these points. The quantity of this steel was two-thirds of that required to take the maximum positive moments in the members. The shears and reactions for continuous members, such as floor-beams and stringers, were increased by from 10 to 25% (depending on the number of spans) more than what these functions would have been for simple spans.

In order to make clear the methods used in the design of the arch ribs, braces, and hinges, a detailed account of this part of the work will be given.

It will be seen that the structural steel arch rib reinforcement takes stress in two independent steps: First, as the ribs are filled with concrete, the members take certain initial stresses, which, as the concrete sets, remain in the steel. This initial stress in the steel was not allowed to exceed a maximum of 12 500 lb. per sq. in. in compression, as given in the list of unit stresses. Second, after the ribs are finished and the construction of the posts and deck is under way, and later, when the structure is carrying live load, stresses are set up in the ribs, which are carried jointly by the steel and concrete in the usual manner. The assumed ratio of the moduli of elasticity of steel and concrete was taken as 15. Therefore, the maximum possible compressive stress in the structural steel reinforcement of the ribs would be 12 500 lb. plus 15 times 500 lb., or 20 000 lb. per sq. in. This is based on the assumption that the ribs and braces are completely filled with concrete in so short a time that it does not begin to set before all the load is in place, and also that the stress in the concrete at the center of gravity of the steel chord is 500 lb. per sq. in. As a matter of fact, the ribs were concreted in sections, and the braces were not concreted until the concrete in the ribs had set for several days. Consequently, the initial compression in the structural reinforcement was much less than that mentioned above. The center of gravity of the steel chords is some distance in from the face of the concrete, so that the concrete

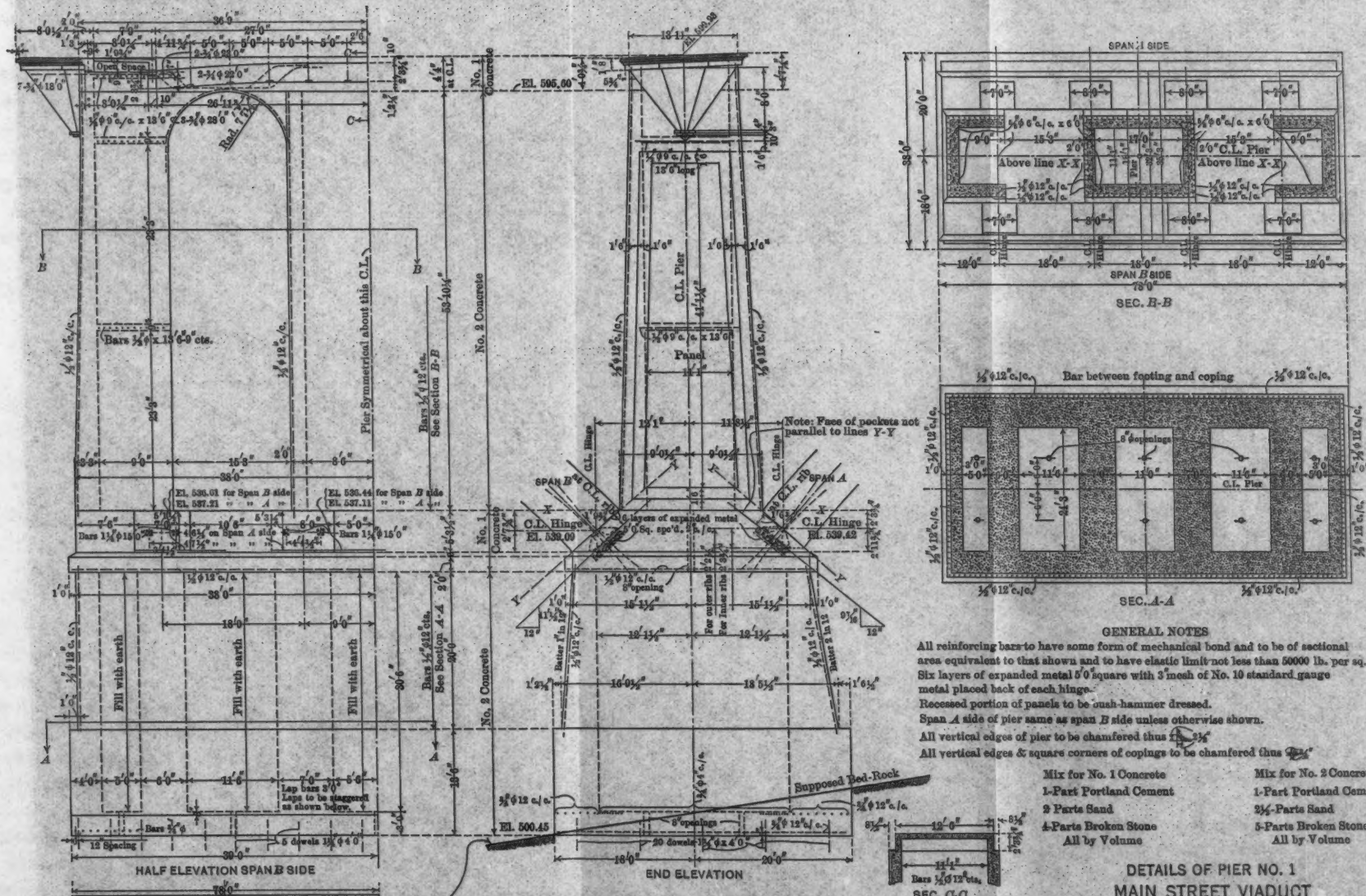
stress at this point, and therefore the stress in the steel, is lower than that assumed. The actual maximum compression in the structural reinforcement is about 16 000 lb. per sq. in.

The lines of pressure for the ribs were worked out graphically for the various positions of the live load, combined with dead load, and for the various stages in the construction of the deck. In order to allow for the weight of the ribs themselves, their dimensions were first assumed, and later were revised where necessary. The neutral axes of the ribs were drawn midway between the extreme pressure lines. This made the neutral axis of the rib a constructive curve, approximating a parabola with its vertex at the crown of the arch. The axis as drawn followed closely the pressure line for full dead and live load. The inner and outer ribs were made of the same vertical dimensions, but of different thickness.

The next step was to find the stresses in the reinforced concrete rib for the various stages in the construction of the deck, and for the dead and live load, under working conditions in the finished structure. The quantity of steel required at each point to keep the extreme fiber stress in the concrete down to 500 lb. per sq. in. for the worst condition, was then computed. The required area of steel in either face of the rib at any point was not allowed to exceed 2.5% of the total cross-sectional area of the rib. Where, on first trial, the required quantity of steel exceeded the above maximum, a re-design was made, using a rib of larger cross-section.

As mentioned under the head of "Loading," the stresses in the structural steel ribs were found graphically for the various positions of load during the concreting of the ribs and braces, using the rib having the dimensions which had been determined as just described. The sections of the structural steel reinforcement were then determined. This structural steel reinforcement, though designed to carry the ribs and braces during their construction, also forms part of the reinforcement of the finished rib, as previously mentioned. Where the required area of steel in the finished rib exceeded that supplied by the structural reinforcement, the deficiency was made up by the addition of reinforcing bars. These bars do not run the full length of the ribs, but stop off where they are not needed. It will be noticed that these bars take little, if any, initial stress, such as is taken by the structural reinforcement. Additional bars are placed in the

PLATE XIX.
PAPERS, AM. SOC. C. E.
AUGUST, 1914.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



GENERAL NOTES

All reinforcing bars to have some form of mechanical bond and to be of sectional area equivalent to that shown and to have elastic limit not less than 50000 lb. per sq. in.
Six layers of expanded metal 5'0" square with 3" mesh of No. 10 standard gauge metal placed back of each hinge.
Recessed portion of panels to be rush-hammer dressed.
Span *A* side of pier same as span *B* side unless otherwise shown.
All vertical edges of pier to be chamfered thus 1" × 2½".
All vertical edges & square corners of copings to be chamfered thus 2" × 2".

Mix for No. 1 Concrete	Mix for No. 2 Concrete
1-Part Portland Cement	1-Part Portland Cement
2 Parts Sand	2½-Parts Sand
4-Parts Broken Stone	5-Parts Broken Stone
All by Volume	All by Volume

NOTE

Bed-rock to be benched out in horizontal steps and footing course of pier to be built to suit these steps as directed by the engineers.

DETAILS OF PIER NO. 1
MAIN STREET VIADUCT
FORT WORTH, TEXAS

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sides of the ribs, running full length. These bars help to resist transverse bending stresses, and to prevent surface cracks from appearing.

The transverse forces of wind and water pressure, mentioned previously, were assumed to be resisted wholly by the ribs and braces acting as a series of portals, in the same manner as the columns and girders of an office building act to resist wind pressure. The deck was not given credit for any resisting power, because of the presence of the expansion joints over the haunch and crown of the span.

Points of inflection were assumed in the braces, midway between each pair of ribs, and in the ribs, midway between the braces. The ribs being hinged at the haunches and crown, were computed as being free to move at these points. Bars were provided at the haunches, however, anchoring the ribs to the piers, and at the crown, anchoring the two halves of the rib together. These bars were placed in the plane of the neutral axis of the rib, and tended to resist transverse, but not vertical movement, of the ribs. The stresses in the ribs and braces were computed at each section, for the moments, shears, and direct thrusts in the members. The maximum stresses at each point in the ribs, due to transverse forces, were combined with the maximum stresses due to the vertical loads, and the ribs were proportioned accordingly.

The design of the hinges naturally received considerable attention. At first, a hinge was studied which consisted of two castings, having semi-cylindrical bearings, and a pin. This type has a number of advantages, but was finally abandoned on account of the difficulty of setting the hinges on the piers with sufficient accuracy. Unless the axes of the pins are placed exactly at right angles to the vertical plane through the longitudinal axis of the rib, eccentric stresses will result in the rib, which may be too large for safety. It was finally decided to use hinges composed of two ribbed steel castings, having a hemispherical ball and socket joint, as shown on the plans. This joint was machined, and, by allowing movement in all directions, made great accuracy in setting the castings unnecessary. This joint also made it possible to adjust the structural rib reinforcement properly in vertical planes, with the temporary adjustable lateral rods provided for that purpose. This type of hinge reduces to a minimum the possibility of eccentric stresses in the ribs.

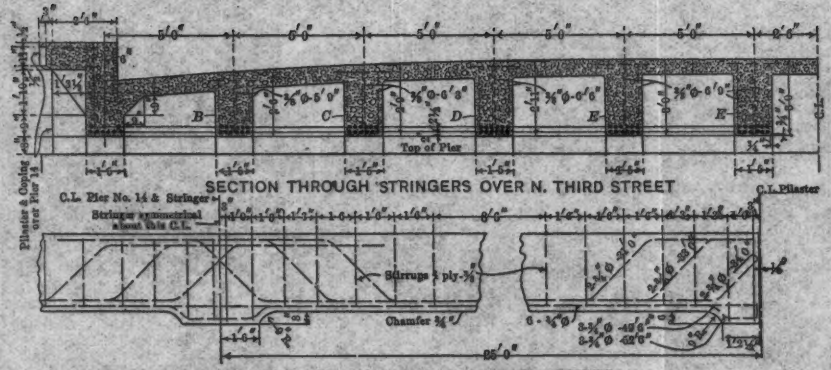
Bearings similar to these hinges had previously been used for some slow-moving multiple trucks, carrying extremely heavy loads. In this case, movement in all directions was necessary to take up the inequalities in the track. These bearings gave perfect satisfaction.

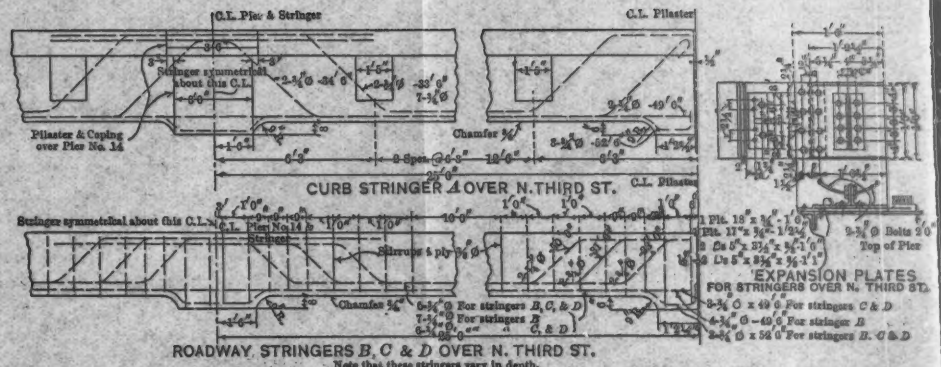
In proportioning the hinge castings, much the same method was followed as in the design of a column base. A section was taken through the diagonal rib of the hinge, at the junction of the rib with the body, or shaft, of the casting, and the tensile, compressive, and shearing stresses on the section were computed. A section was also taken through the center of the hinge, and the stresses on this section were computed, on the assumption that the reaction on the base of the casting produced moments at right angles to this section, instead of radially, as had previously been assumed. The resultant of the radial and direct compressive stresses at the circumference of the shaft of the casting, was also investigated.

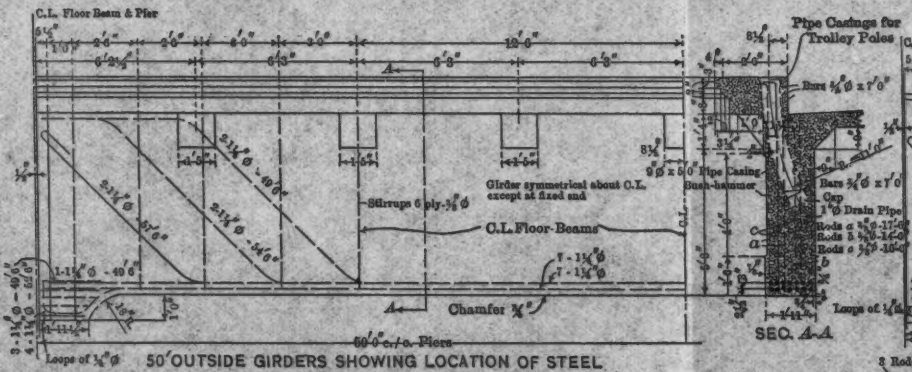
The camber of the arch spans was computed for full dead and live load, with impact, combined with temperature and an allowance for shrinkage. The rise of the structural ribs was increased by the amount of the camber. The total calculated camber for the longest span was about $3\frac{1}{2}$ in.

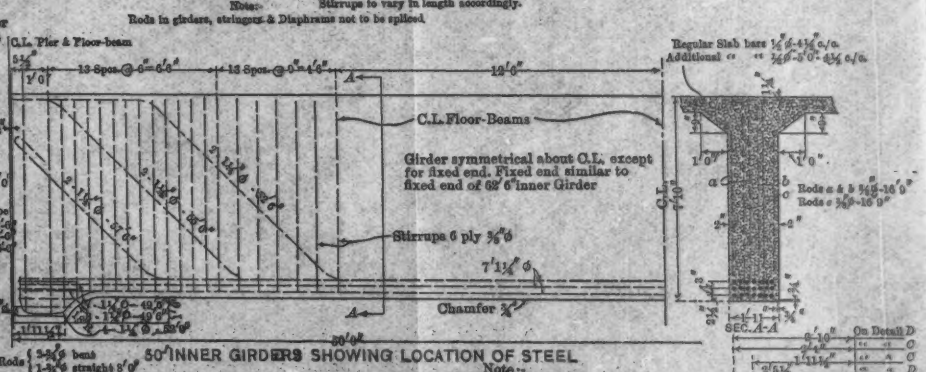
In the design of the main piers, considerable attention was given to the question of protection against scour. Pier No. 2 was given particular attention because of the fact that it is only a short distance below a power dam having a fall of about 15 ft. The bottom of the footing course was placed at about 5 ft. below the bed of the stream. Oak piles were provided, extending about 15 ft. below the bottom of the footing. The sheet-piling for the coffer-dam was driven about 8 ft. below the bottom of the footing, and was left in place. The banks of the stream at this point were also protected by a large quantity of rip-rap. These precautions, together with the fact that the material in the river bed at this point is a stiff clay not likely to erode, were believed to afford ample protection against scour.

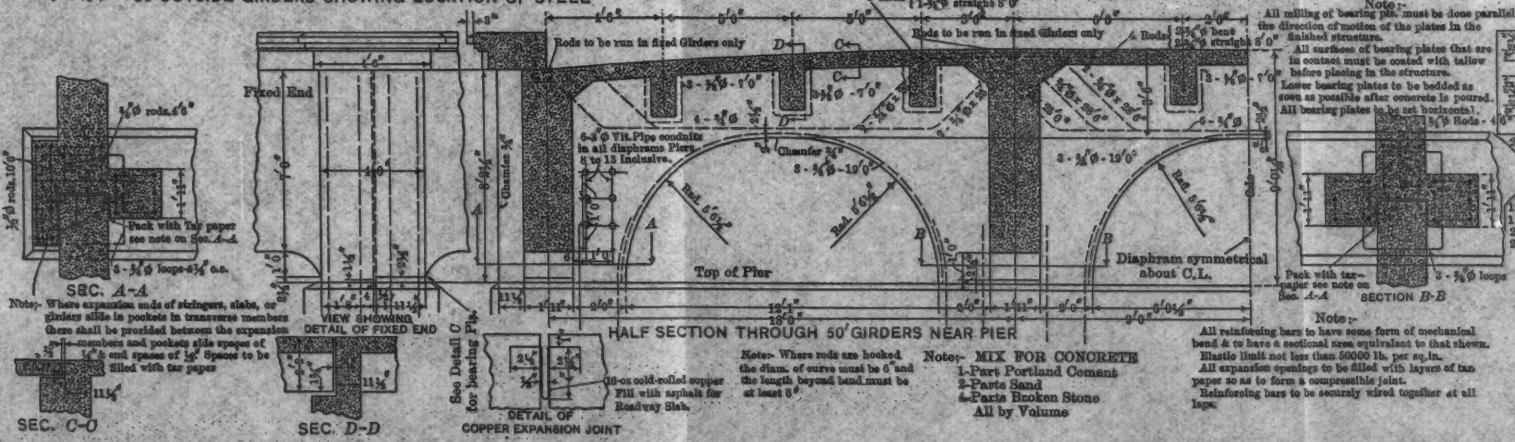
Several different designs and estimates were made for Pier No. 2 before a satisfactory solution was reached. The following were considered: First, a pier similar to Pier No. 1, founded on bed-rock; second, a pier having its base at about the level of the bed of the river, with the vertical loads carried on eight cylinder piers, two under each line of arch ribs; third, a pier resting on piles driven to rock;











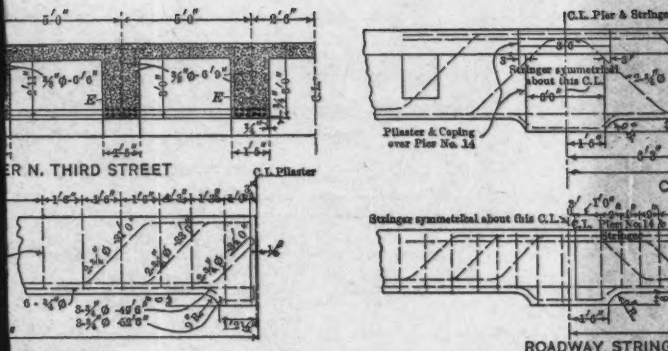
Note:- Where expansion ends of stringers, slabs, or girders slide in pockets in transverse members there shall be provided between the expansion members and pockets side spaces of ½" and space of ½" Spaces to be filled with tar paper

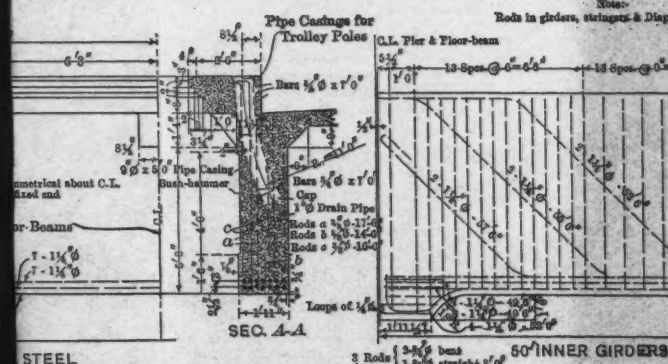
Note:- Where rods are hooked the diam. of curve must be 6" and the length beyond bend must be at least 6".

Note:- MIX FOR CONCRETE
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume

All reinforcing bars to have some form of mechanical bond & to have a sectional area equivalent to that shewn.
Elastic limit not less than 50000 lb. per sq. in.
All expansion openings to be filled with layers of tar paper so as to form a compressible joint.
Reinforcing bars to be securely wired together at all laps.

50' GIRDER DETAILS
MAIN STREET VIADUCT
FORT WORTH, TEXAS





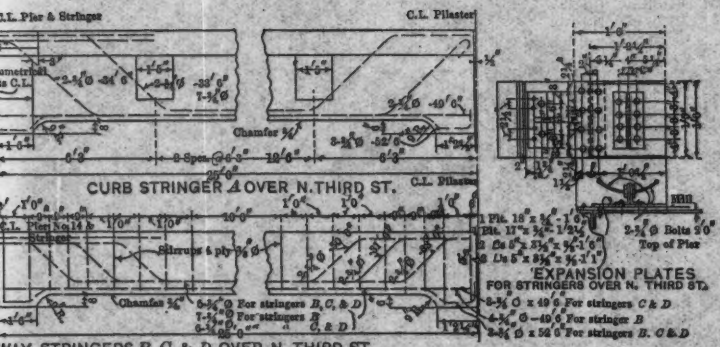
16-oz cold-rolled copper
Fill with asphalt for Roadway Slab.

AIL OF
PANSION JOINT

Note:- Where rods are hooked the diam. of curve must be 6" and the length beyond bend must be at least 6"

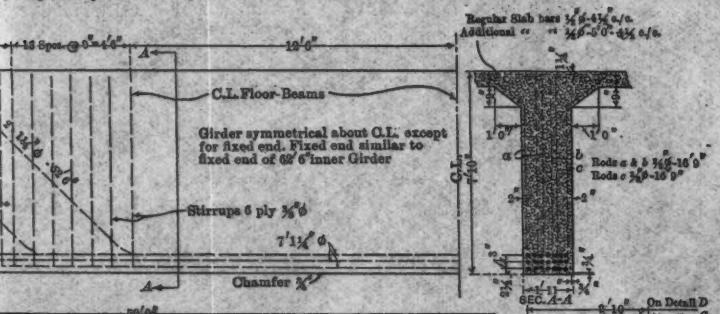
Note:- MIX FOR CONCRETE
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume

PLATE XX.
PAPERS, AM. SOC. C. E.
AUGUST, 1914.
BOWEN ON
REINFORCED CONCRETE VIADUCTS.



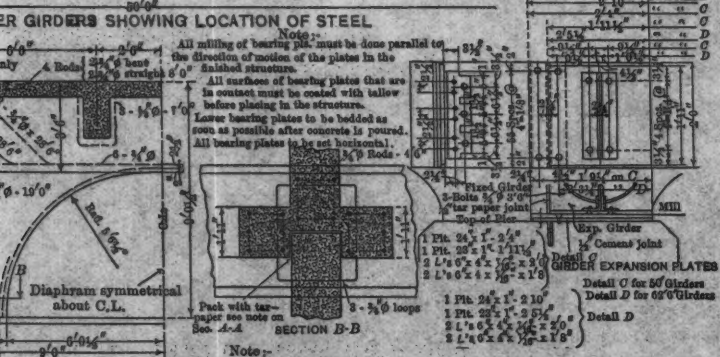
WAY STRINGERS B, C & D OVER N. THIRD ST.

Note:- Note that these stringers vary in depth.
Stirrups to vary in length accordingly.
s, stringers & Diaphrams not to be spliced.



ER GIRDERS SHOWING LOCATION OF STEEL

Note:-
All milling of bearing plts. must be done parallel to the direction of motion of the plates in the finished structure.
All surfaces of bearing plates that are in contact must be coated with tallow before placing in the structure.
Lower bearing plates to be bedded as soon as possible after concrete is poured.
All bearing plates to be set horizontal.



Note:-
All reinforcing bars to have some form of mechanical bend & to have a sectional area equivalent to that shown.
Elastic limit not less than 50000 lb. per sq. in.
All expansion openings to be filled with layers of tar paper so as to form a compressible joint.
Reinforcing bars to be securely wired together at all laps.

CRETE
ment
ne

50'GIRDER DETAILS
MAIN STREET VIADUCT
FORT WORTH, TEXAS

and fourth, the pier as built, having moderately spread footings, but with a portion of the load carried on piles which do not extend to rock.

The first scheme was abandoned because of the cost, which would have made that of the structure exceed the appropriation. The second scheme was considered inadvisable because of the difficulty of taking care of the unbalanced horizontal thrust. As the vertical load would have been carried down the cylinders to rock, the friction between the pier base and the material under it could not safely be counted on to resist the horizontal thrust. There then remained, to resist this thrust, only the pressure of the earth against the body of the pier and against the cylinders. Computations indicated that this resisting pressure would be insufficient, unless the body of the pier was extended to a considerably greater depth. This would have made the cost too great. The third scheme was rejected for practically the same reasons as those given above for the second scheme, and also because, after investigation, it was considered doubtful whether piles could be driven to rock through the mixture of sand and gravel overlying it. The fourth scheme proved to be the cheapest of those considered, and was believed to be the most feasible. Therefore it was adopted, with the provisions mentioned above to protect the pier from scour.

The small piers were designed for full vertical load only, using the unit pressures mentioned previously.

Rankine's theory of earth pressure was used in the design of the retaining walls. The weight of the earth fill was taken at 100 lb. per cu. ft. A superposed load of 100 lb. per sq. ft. was assumed, to take care of the roadway and sidewalk loads. For convenience in computing the pressures, the unit weight of the fill was considered to be increased by the ratio, $\frac{d'}{d}$, where d is the actual depth of fill, and d' is the actual depth plus the depth of the imaginary fill required to produce the same unit vertical pressure as the superposed load. In the foregoing case, $d' = d + 1$ ft. For the higher sections, this increase is negligible, but, for the lower ones, it is worth taking into account.

In concluding this portion of this paper, a few words in regard to the advantages of the three-hinged, ribbed arch may be in order.

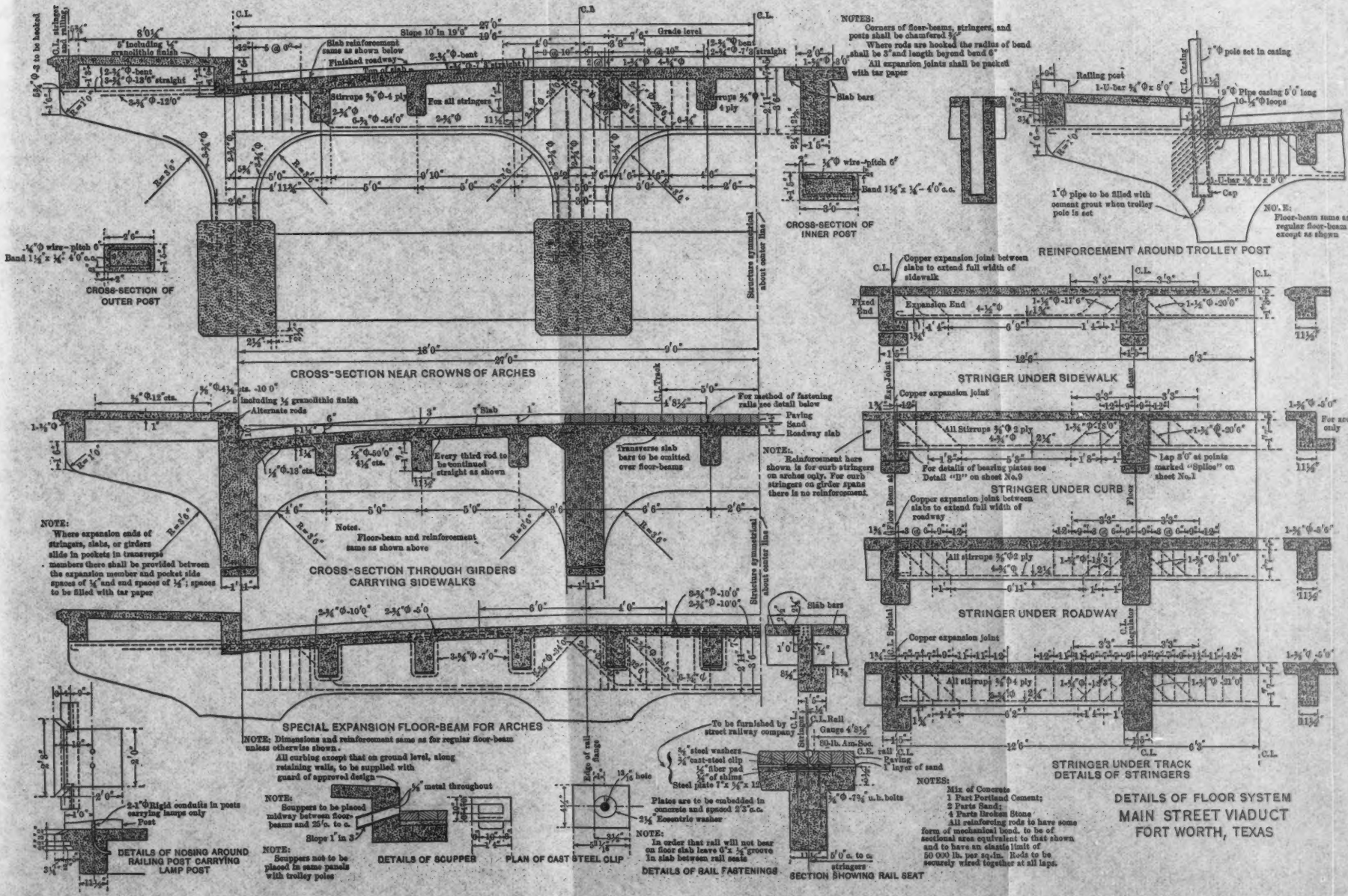
So much has been said on this subject, that little that is new can be brought out. However, to go over the ground again briefly, it is evident that the stresses in an arch of this type are statically determinate, and can be computed with as much accuracy as the loading assumptions and computations of the dead weight warrant. Temperature stresses and those due to settlement are eliminated, and the computations, though somewhat long, are simple. Practically, the only disadvantage is the cost of the hinges. In the Main Street Viaduct this item amounted to about 3½% of the total cost. This is more than it should have been, owing to the high price paid for the steel castings. Even at this price, when all the advantages of the three-hinged arch are considered, the money paid for hinges is well spent. The ribbed arch has two great advantages over the barrel, or solid arch. These are lightness, with consequent saving in cost, and the fact that water-proofing is not required. The latter, alone, is enough to recommend it to those who have had trouble with this feature of arch construction.

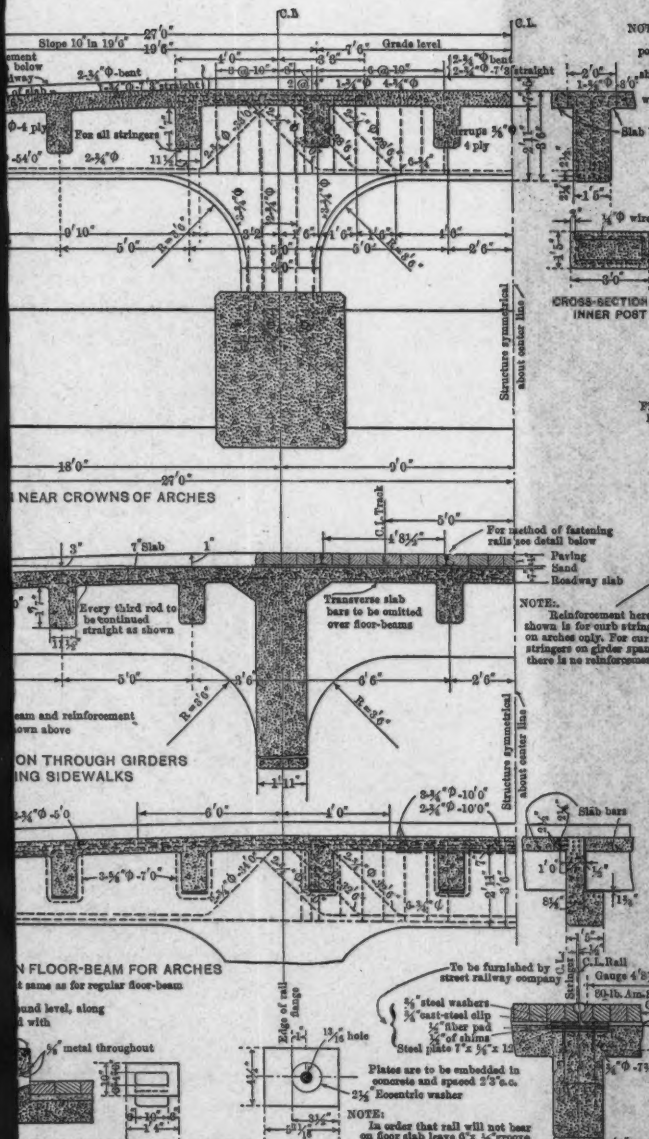
The increasing use of the three-hinged ribbed arch, both in the United States and in Europe, is evidence of its many advantages.

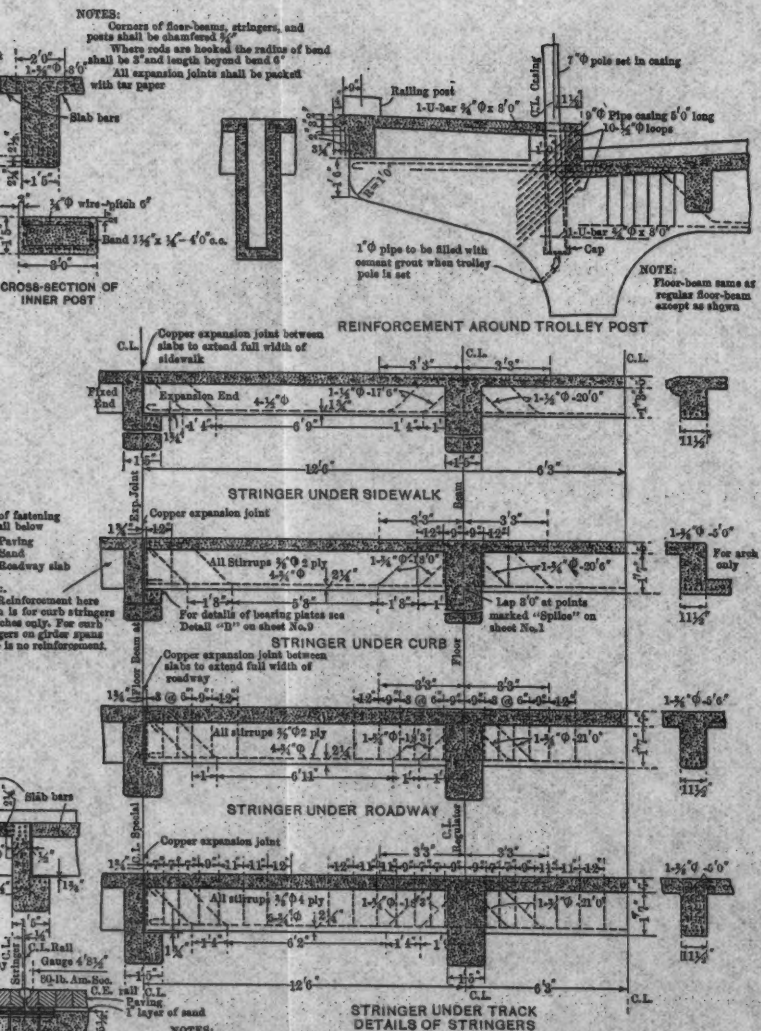
Construction.

Contractor's Plant and Equipment.—Work on the Main Street Viaduct started early in December, 1912, and was finished in March, 1914. The climate of Fort Worth is such that concreting can be carried on at all seasons of the year. The only interruptions were those due to rains, which were infrequent.

The general contractor's plant and equipment consisted of a central mixing plant, two distributing towers, with hoists, chutes, etc., an electric locomotive and cars for transporting concrete, derricks and clam-shell buckets for handling excavation and materials, a pair of driver leads suspended from the boom of a derrick, and the usual equipment of smaller tools and appliances. The sub-contractor for the structural steelwork had a steel stiff-leg derrick, a derrick car, two gin-poles, a two-bent timber gantry traveler, and the necessary hoisting engines and minor appliances. With the exception of one hoisting engine, all the general contractor's equipment was electrically driven, current being obtained from the local power com-


CROSS-SECTION NEAR CROWNS OF ARCHES
CROSS-SECTION THROUGH GIRDERS CARRYING SIDEWALKS
SPECIAL EXPANSION FLOOR-BEAM FOR ARCHES
CROSS-SECTION OF OUTER POST
CROSS-SECTION OF INNER POST
REINFORCEMENT AROUND TROLLEY POST
STRINGER UNDER SIDEWALK
STRINGER UNDER CURB
STRINGER UNDER ROADWAY
STRINGER UNDER TRACK
DETAILS OF STRINGERS
DETAILS OF NOSING AROUND RAILING POST CARRYING LAMP POST
DETAILS OF SCUPPER
PLAN OF CAST STEEL CLIP
DETAILS OF RAIL FASTENINGS
SECTION SHOWING RAIL SEAT
NOTES:
Corners of floor-beams, stringers, and posts shall be chamfered ¾".
Where rods are hooked the radius of bend shall be 3" and length beyond bend 6"
All expansion joints shall be packed with tar paper
NOTE:
Where expansion ends of stringers, slabs, or girders slide in pockets in transverse members there shall be provided between the expansion member and pocket side spaces of ¼" and end spaces of ½"; spaces to be filled with tar paper
NOTE:
Reinforcement here shown is for curb stringers on arches only. For curb stringers on girder spans there is no reinforcement.
NOTE:
Dimensions and reinforcement same as for regular floor-beam unless otherwise shown.
All curbing except that on ground level, along retaining walls, to be supplied with guard of approved design
NOTE:
Scuppers to be placed midway between floor-beams and 25'0" c. to c.
Slope 1" in 3"
NOTE:
Scuppers not to be placed in same panels with trolley poles
NOTE:
In order that rail will not bear on floor slab leave 6"x ½" groove in slab between rail seats
NOTES:
Mix of Concrete
1 Part Portland Cement;
2 Parts Sand;
4 Parts Broken Stone
All reinforcing rods to have some form of mechanical bond, to be of sectional area equivalent to that shown and to have an elastic limit of 50 000 lb. per sq.-in. Rods to be securely wired together at all laps.
DETAILS OF FLOOR SYSTEM
MAIN STREET VIADUCT
FORT WORTH, TEXAS


C.L.
Slope 10 in 19'6"
27'0"
19'6"
Grade level
4'0"
3'0"
7'6"
2-¾"Φ bent
2-¾"Φ straight
Φ-4 ply
For all stringers
Stirrups ⅜"Φ 4 ply
2-¾"Φ
11½"
0'10"
5'0"
R=16'
6'0"
1'6"
2'6"
Slab
CROSS-SECTION
INNER POST
Structure symmetrical about center line
18'0"
27'0"
NEAR CROWNS OF ARCHES
C.L. Track
5'0"
4'8½"
For method of fastening rails see detail below
3"
7" Slab
1"
Paving
Sand
Roadway slab
Every third rod to be continued straight as shown
Transverse slab bars to be omitted over floor-beams
NOTE:
Reinforcement here shown is for curb string on arches only. For cur stringers on girder span there is no reinforcemen
5'0"
3'6"
6'6"
2'0"
R=36'
R=32'
ON THROUGH GIRDERS
ING SIDEWALKS
Structure symmetrical about center line
2-¾"Φ -10'0"
2-¾"Φ -10'0"
Slab bars
6'0"
4'0"
2-¾"Φ -5'0
2-¾"Φ -7'0"
1'0"
8¼"
15⅛"
5"
C.L. Rail
Stringer
Gauge 4'8
80-lb. Am.
FLOOR-BEAM FOR ARCHES
To be furnished by street railway company
Edge of rail flange
⅝" steel washers
¾" cast-steel clip
⅛" fiber pad
⅛" of shims
Steel plate 1"x ¾"x 12
13/16" hole
⅝" metal throughout
Plates are to be embedded in concrete and spaced 2'3" c.c.
2½" Eccentric washer
NOTE:
In order that rail will not bear on floor slab leave ⅛" groove in slab between rail seats
TAILS OF SCUPPER
PLAN OF CAST STEEL CLIP
DETAILS OF RAIL FASTENINGS
SECTION SHOWI



NOTES:

Mix of Concrete
1 Part Portland Cement;
2 Parts Sand;
4 Parts Broken Stone

All reinforcing rods to have some form of mechanical bond, to be of sectional area equivalent to that shown and to have an elastic limit of 50 000 lb. per sq.in. Rods to be securely wired together at all laps.

DETAILS OF FLOOR SYSTEM
MAIN STREET VIADUCT
FORT WORTH, TEXAS

UNIV. OF MICH.

pany and transformed at the site. The sub-contractor's equipment was operated by steam.

While the pier excavations were under way, the mixing plant was being built near Pier No. 3 on the down-stream side of the structure. This consisted of an elevated hopper divided into two parts, one for rock and one for sand, a measuring box for sand and one for stone, and a mixer capable of handling a batch of a little more than 1 cu. yd. of concrete. Behind the mixing plant was a stiff-leg bull-wheel derrick for unloading materials and for supplying the hoppers of the mixing plant. Materials were brought to the site on cars, on a spur-track which crossed the line of the viaduct alongside of the mixing plant.

Concrete was discharged from the mixer into steel hopper cars drawn by an electric locomotive running on a narrow-gauge track. The concrete was carried in these cars to the two distributing towers, where it was hoisted and conveyed through the chutes to its destination. The larger of these towers was stationary, and was about 200 ft. high. This tower served the portion of the viaduct, between Pier No. 4 and the South Abutment, except where concrete was discharged directly into the footings of Pier No. 3 from the mixer. The other and smaller tower was built so that it could be moved when required. It served the portion of the structure between Pier No. 4 and the North Abutment. For the retaining walls, concrete was carried in the hopper cars to the north end of the walls, where it was switched back on a trestle midway between the walls, and having the track parallel to and about 6 ft. above the tops of the walls. From this trestle the concrete was passed through chutes into the walls.

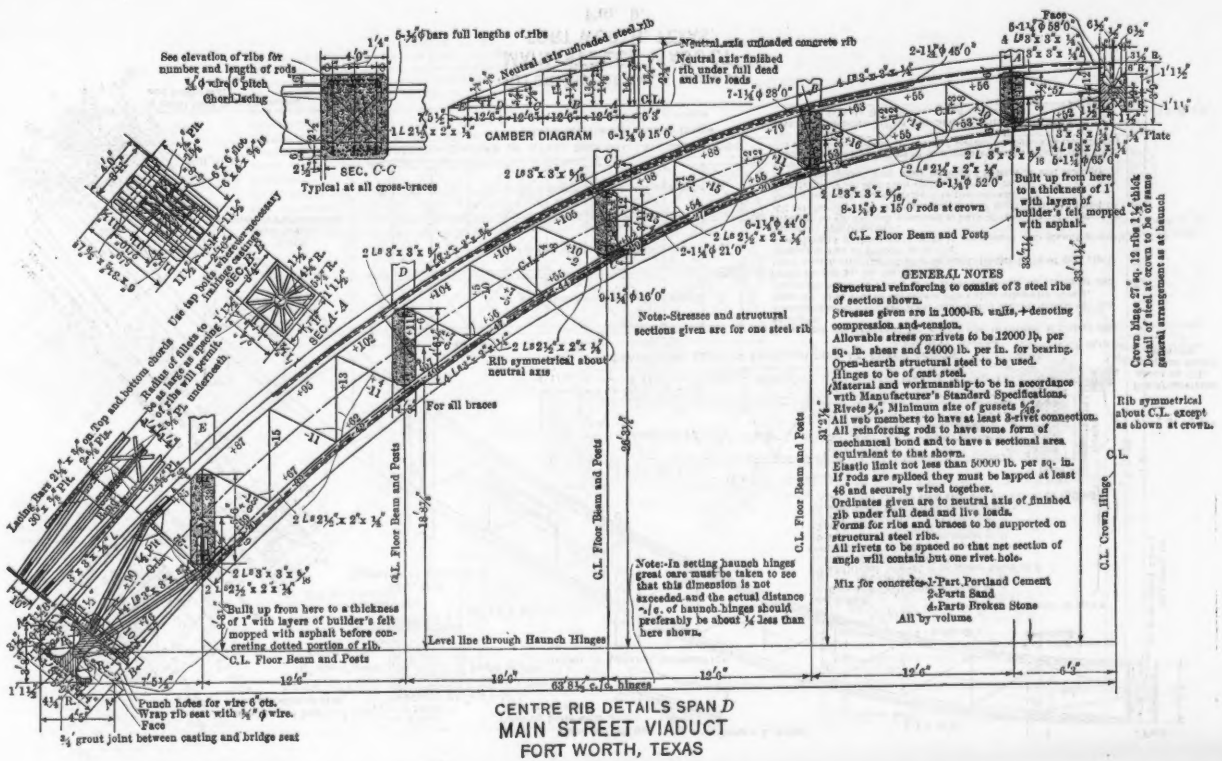
Construction Methods.—Steel forms were used where there was sufficient duplication to make them economical. They were especially advantageous on the girders and floor system, which had been laid out with the view of making the work as simple, and giving as much repetition, as possible. For instance, all floor-beams were made alike, so far as the dimensions of the concrete were concerned; all panel lengths, and consequently all stringers of each kind, were alike. The wall forms were made up of squares and rectangles of such size that they could be handled readily by two or three men. These forms were stiffened by angles, and were bolted together by bolts passing through the flange angles. They were braced by timber verticals and

shores. The grades of the walls, coping, and any irregular portions were formed in wood. Sufficient deck and girder forms were provided to make two of the 62 ft. 6-in. girder spans. These forms were reinforced in order to prevent bulging and deflection between panel points.

The resulting surface finish was in general very good, and required little working, after the forms were removed, to produce the desired results. On removing the forms, the surface was bush-hammered, and all fins and lips left at the junction of the form sections were removed. In some cases where the forms had been insufficiently braced untrue surfaces were corrected by bush-hammering. Where necessary, the surface was rubbed down with carborundum blocks, and washed with neat cement grout, which was rubbed into the work. As a result of this treatment the surfaces present a neat and uniform appearance, which is very pleasing. Steel forms were used for the railings. The panels and hand-rails were cast on the ground, and erected; the posts and foot-rails were cast in place. Particular attention was given to the surface finish of the railings. All surfaces which would be exposed in the finished work, were rubbed down with carborundum blocks as soon as possible after the removal of the forms. The work was then treated with a thin coat of neat cement grout, well rubbed in, and the surplus grout washed off. This process was repeated until the surface pores were filled, and the work presented a smooth uniform surface.

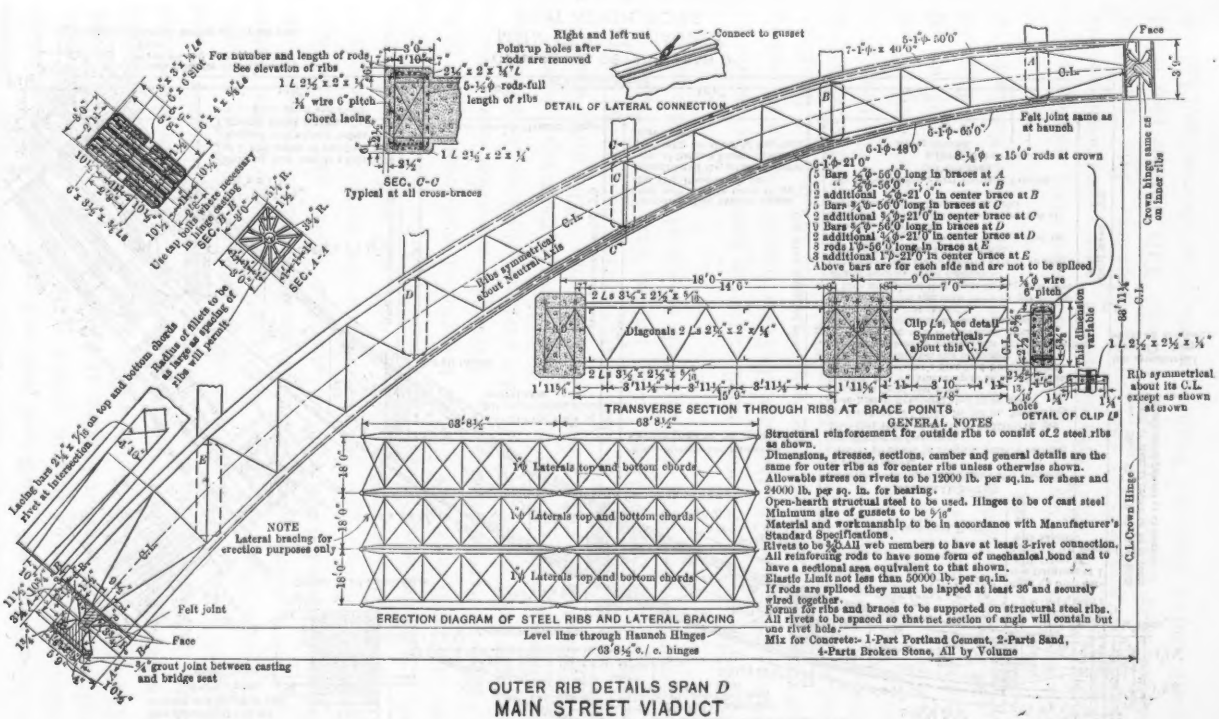
After the main piers were finished to the tops of the skewbacks indicated on the plans, by the lines, *X-X*, the hinges were set in pockets provided for that purpose. The space between the base of the hinge casting and the back of the pocket, of about $\frac{3}{4}$ in., was filled with neat cement grout, and the pocket was filled with concrete to the line, *Y-Y*, shown on the plans, leaving only the socket of the hinge exposed. After the concrete had become thoroughly set, the erection of the structural steel reinforcement was commenced. This was done before finishing the piers above the lines, *X-X*, in order to facilitate the handling of the erection equipment.

Two methods were used in erecting this structural reinforcement. Spans *C*, *D*, and the south arm of Span *A*, were raised with derricks and gin-poles, and without the use of falsework. For Span *B*, a two-bent timber gantry traveler was used. This traveler also raised the north arm of Span *A*. Span *D*, which is the shortest and lightest



CENTRE RIB DETAILS SPAN *D*
MAIN STREET VIADUCT
FORT WORTH, TEXAS

FIG. 8.



OUTER RIB DETAILS SPAN *D*
MAIN STREET VIADUCT
FORT WORTH, TEXAS.
FIG. 9.

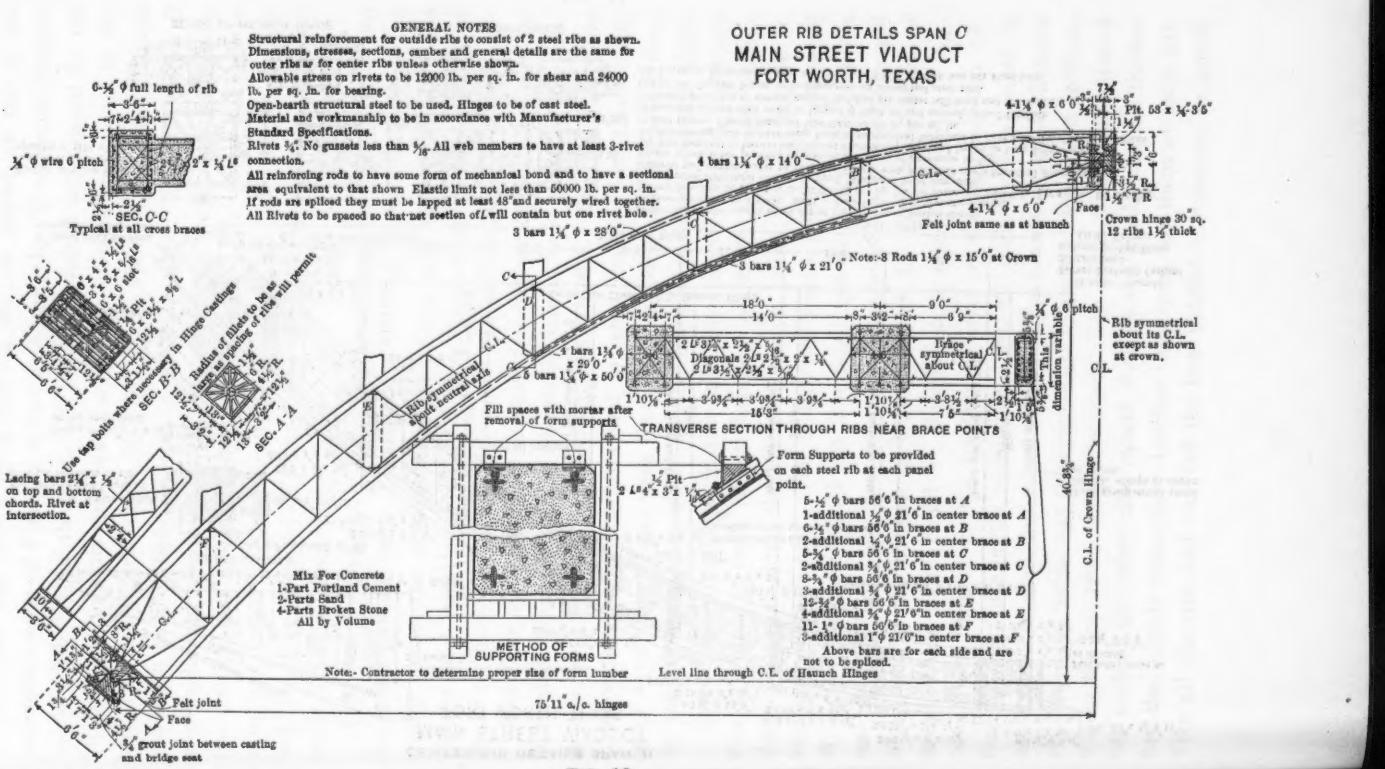
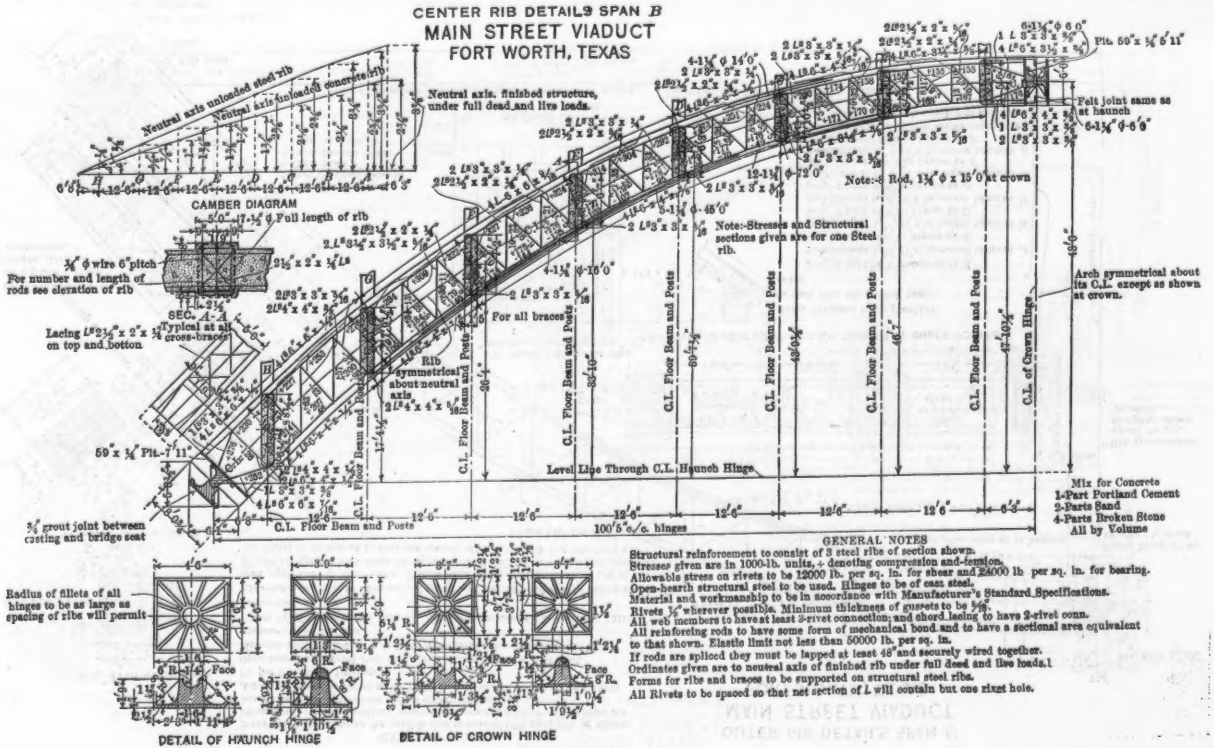

GENERAL NOTES
Structural reinforcement for outside ribs to consist of 2 steel ribs as shown. Dimensions, stresses, sections, camber and general details are the same for outer ribs as for center ribs unless otherwise shown.
Allowable stress on rivets to be 12000 lb. per sq. in. for shear and 24000 lb. per sq. in. for bearing.
Open-hearth structural steel to be used. Hinges to be of cast steel.
Material and workmanship to be in accordance with Manufacturer's Standard Specifications.
Rivets ¾". No gussets less than 5/16. All web members to have at least 3-rivet connection.
All reinforcing rods to have some form of mechanical bond and to have a sectional area equivalent to that shown. Elastic limit not less than 50000 lb. per sq. in. If rods are spliced they must be lapped at least 48" and securely wired together. All Rivets to be spaced so that net section of L will contain but one rivet hole.
OUTER RIB DETAILS SPAN C
MAIN STREET VIADUCT
FORT WORTH, TEXAS
6-¾" ϕ full length of rib
¼" ϕ wire 6" pitch
SEC. C-C
Typical at all cross braces
SEC. B-B
SEC. A-A
Use top bolts where necessary in Hinge Castings
Radius of fillets to be as large as spacing of rivets will permit
Lacing bars 2¼" x ½" on top and bottom chords. Rivet at intersection.
Mix For Concrete
1-Part Portland Cement
2-Parts Sand
4-Parts Broken Stone
All by Volume
Fill spaces with moriar after removal of form supports
Rib symmetrical about neutral axis
METHOD OF SUPPORTING FORMS
Note:- Contractor to determine proper size of form lumber
Felt joint
Face
¾" grout joint between casting and bridge seat
4 bars 1¾" ϕ x 14'0"
3 bars 1¾" ϕ x 28'0"
3 bars 1¼" ϕ x 21'0"
Note:-8 Rods 1¼" ϕ x 15'0" at Crown
4 bars 1¾" ϕ x 29'0"
5 bars 1¾" ϕ x 50'0"
4-1½" ϕ x 6'0"
Felt joint same as at haunch
Face
Crown hinge 30" sq. 12 ribs 1½" thick
Plt. 53" x ¾" x 3'6"
Diagonals 2 L 3½" x 2½" x ⅜"
Rib symmetrical about its C.L. except as shown at crown.
TRANSVERSE SECTION THROUGH RIBS NEAR BRACE POINTS
Form Supports to be provided on each steel rib at each panel point.
5-½" ϕ bars 56'6" in braces at A
1-additional ½" ϕ 21'6" in center brace at A
6-½" ϕ bars 56'6" in braces at B
2-additional ½" ϕ 21'6" in center brace at B
5-¾" ϕ bars 56'6" in braces at C
2-additional ¾" ϕ 21'6" in center brace at C
8-¾" ϕ bars 56'6" in braces at D
3-additional ¾" ϕ 21'6" in center brace at D
12-¾" ϕ bars 56'6" in braces at E
4-additional ¾" ϕ 21'6" in center brace at E
11-1" ϕ bars 50'6" in braces at F
3-additional 1" ϕ 21'0" in center brace at F
Above bars are for each side and are not to be spliced.
Level line through C.L. of Haunch Hinges
75'11" c/c. hinges
C.L. of Crown Hinge

FIG. 12.



GENERAL NOTES

Structural reinforcement to consist of 3 steel ribs of section shown.
Stresses given are in 1000-lb. units, + denoting compression and—tension.
Allowable stress on rivets to be 12000 lb. per sq. in. for shear and 24000 lb per sq. in. for bearing.
Open-hearth structural steel to be used. Hinges to be of cast steel.
Material and workmanship to be in accordance with Manufacturer's Standard Specifications.
Rivets ⅞" wherever possible. Minimum thickness of gussets to be ⅝".
All web members to have at least 3-rivet connection and chord lacing to have 2-rivet conn.
All reinforcing rods to have some form of mechanical bond and to have a sectional area equivalent to that shown. Elastic limit not less than 50000 lb. per sq. in.
If rods are spliced they must be lapped at least 48" and securely wired together.
Ordinates given are to neutral axis of finished rib under full dead and live loads.
Forms for ribs and braces to be supported on structural steel ribs.
All Rivets to be spaced so that net section of L will contain but one rivet hole.

FIG. 11.

of the four, was raised by a stiff-leg derrick placed on top of the skewback course of Pier No. 3, and by a gin-pole. The derrick handled the south halves of the ribs, and the north halves were raised by the gin-pole. After this span was finished, the derrick was moved to the top of the skewback course of Pier No. 2, where it was used to raise the south half of Span *C*, and to erect the gantry traveler. Span *C* is considerably heavier than *D*, and, consequently, in raising the ribs, the derrick had to be assisted by a gin-pole. The north half of the span was raised at the same time as the south half, by the derrick and the other gin-pole. The half ribs of both these spans were necessarily handled in one piece.

Low and comparatively light falsework was used for Span *B* and the north half of Span *A*. This was built at about the level of the coping under the skewbacks of the piers, and served to carry the traveler and such bents as were required to support the sections of the ribs. Each half rib was raised in two sections, and field-riveted.

When the steel had been erected, the ribs were carefully lined up in vertical planes with the adjustable lateral rods provided for that purpose, and the field connections were riveted. The next step was to place the reinforcing bars and to wrap the ribs and braces with heavy wire. The forms were then built in place, and supported from the structural reinforcement, in the manner indicated on the plans.

Span *D* was concreted in the following manner: Radial bulkheads were provided in each rib at distances of 22 ft. from each haunch hinge, measured along the rib, and also at points in each rib, 10 ft. from the crown hinge. The transverse braces were also bulkheaded off, so that concrete would not flow into them from the ribs. Chutes were arranged in such a manner that concrete could be discharged in any one of the haunch or crown sections by changing a series of gates. These gates could be changed at a moment's notice. Concreting was started in the haunch sections of the down-stream rib. Four batches, of slightly more than 1 cu. yd. each, were deposited in each haunch section, and alternating batches between the two haunch sections. One batch was then deposited in each of the crown sections, to keep the rib from being distorted. The same order was followed with each of the other ribs, except that, as the inside ribs are thicker than the outside ones, more batches were used for them at each lift. When all the ribs had received the first batches of concrete, as noted,

the process was repeated until the haunch and crown sections had been filled. Lagging was carried up the back of the ribs as the concreting progressed, to hold the concrete, which was necessarily mixed quite wet, in order to work well around the reinforcement. After setting for about 4 days, the bulkheads in the ribs were removed, the surface of the concrete was roughened up, and the closure sections of the ribs were concreted in about the same manner as the haunch sections. The transverse braces were not concreted until the rib concrete had set for several days.

In order to keep track of the distortion of the ribs while concreting was in progress, sash weights were suspended from the steel brace reinforcement close to each rib, at four points along the rib. One weight was placed at the brace on each side of the crown hinge, and one at each quarter point of the span. These weights were suspended by small wires, and carried scales which were wired to the weights. A stake was driven in the ground alongside of each weight, and a nail in the stake acted as a pointer on the scale. The initial reading was taken at each point, and another set of readings was made after each round of concreting. At first a slight settlement of the crown points was observed, followed by a slight rise. The distortion was less than had been anticipated, amounting to a maximum of about $\frac{1}{4}$ in.

The concrete in the ribs and braces was allowed to set for several days before the work of building the forms for the posts and deck of the span was begun. This portion of the work was concreted in the manner indicated under the head of "Loading." The posts were concreted, beginning at the haunches and working toward the crown. Then the deck was concreted from the crown back toward the haunches, with the exception of the middle section, over the crown, which was placed last, in order to provide for the expansion joint at the crown. The piers, in the meantime, had been finished to their full height, in order to insure stability and provide for the connection of the deck to the piers.

The concreting of the other spans was handled in practically the same manner as that just described. When the concreting of a section was started, it was carried through continuously, by working day and night if necessary, so that the work was made as nearly monolithic as possible.

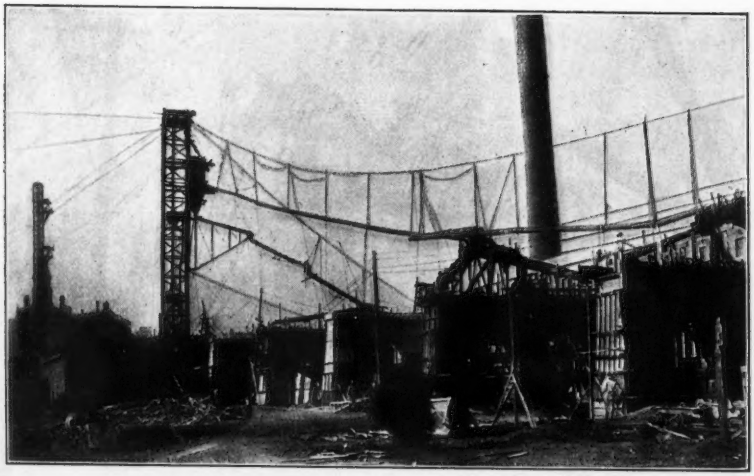


FIG. 12.—MAIN STREET VIADUCT: MOVABLE TOWER AND ARRANGEMENT OF CHUTES FOR CONCRETING PIERS BETWEEN PIER 4 AND NORTH ABUTMENT.

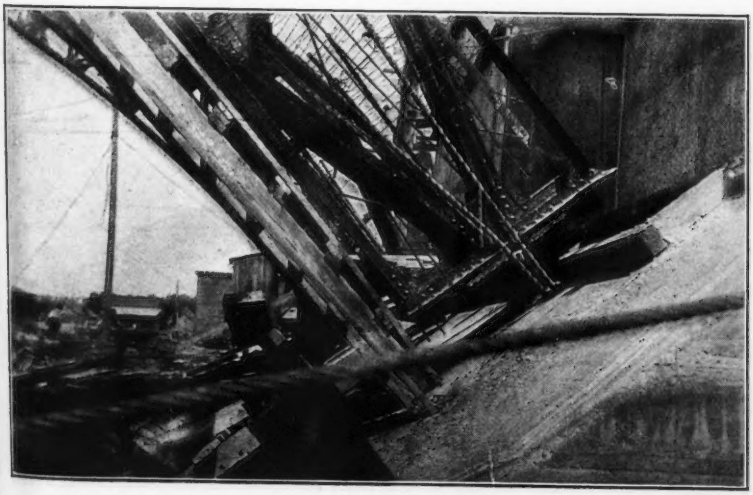


FIG. 13.—MAIN STREET VIADUCT: HAUNCH HINGE OF WEST RIB OF SPAN D. REINFORCING BARS BEING PLACED.

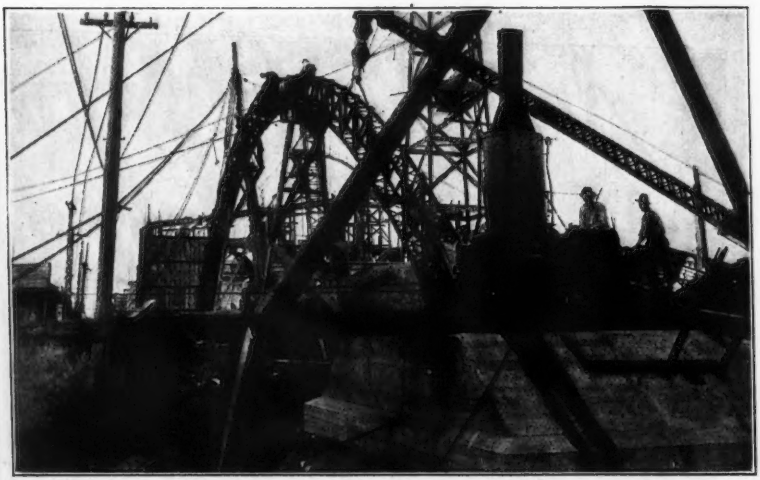


FIG. 14.—MAIN STREET VIADUCT: RAISING OF FIRST STRUCTURAL RIB. WEST RIB OF SPAN D RAISED WITH DERRICK AND GIN-POLE.

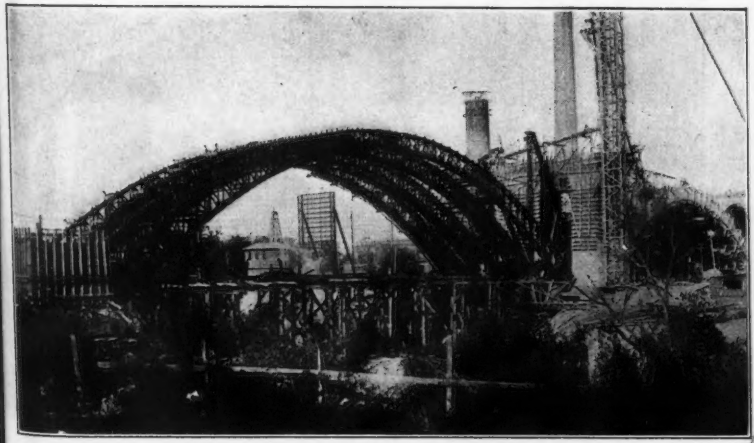


FIG. 15.—MAIN STREET VIADUCT, SPAN B: STEEL ARCHES ERECTED.

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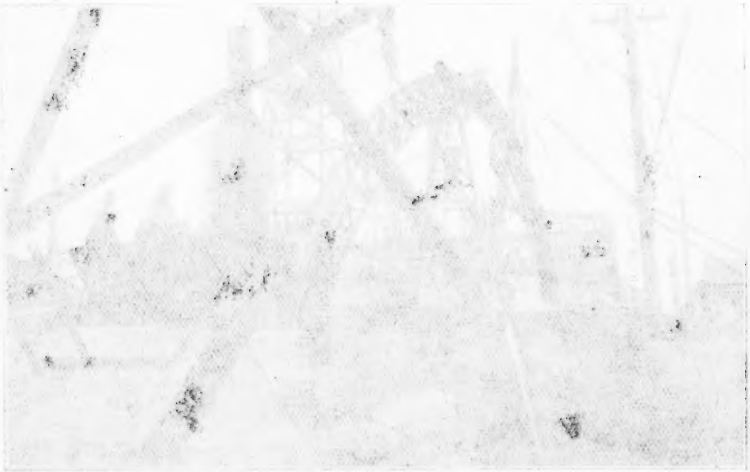

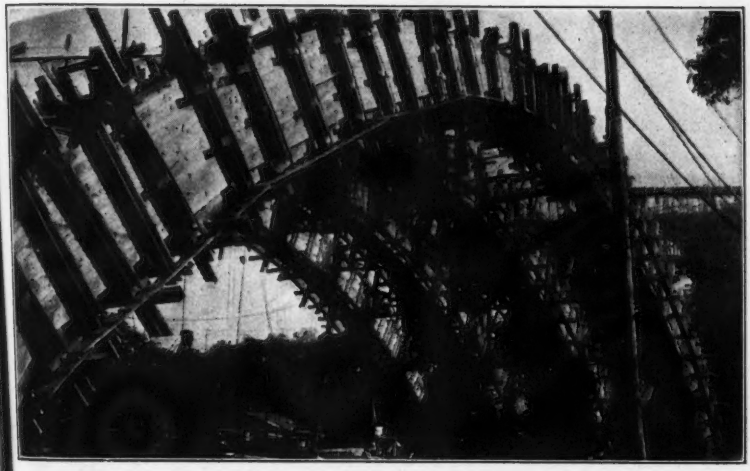

FIG. 16.—MAIN STREET VIADUCT, SPAN C: SHOWING RIBS COMPLETELY FORMED.

FIG. 17.—MAIN STREET VIADUCT: FORMING DECK OF SPAN C.

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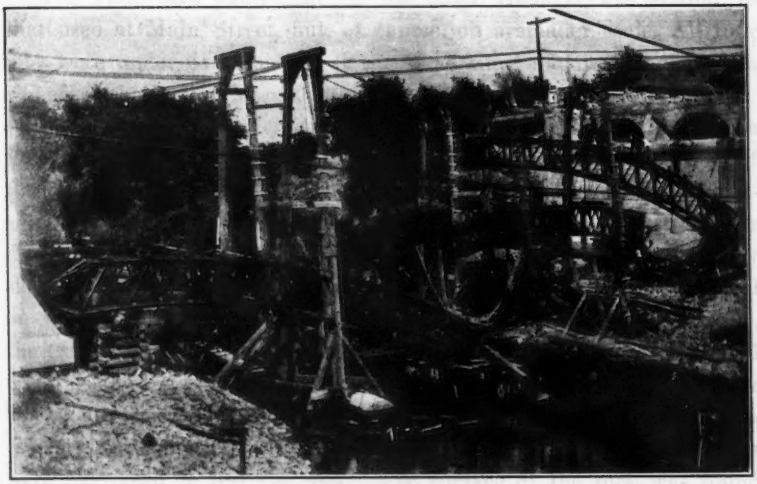

FIG. 18.—WEST SEVENTH STREET VIADUCT: RAISING EAST HALF OF SOUTH RIB OF ARCH BEFORE THE HIGH WATER.

FIG. 19.—ARCH RIBS AFTER THE WATER WENT DOWN, WEST SEVENTH STREET VIADUCT.

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The plant and equipment on the other viaducts was similar to that used at Main Street, but, of course, on a smaller scale. All four jobs were under construction at the same time, and were finished on or before the last of March, 1914.

Difficulties Encountered.—There was some difficulty, at Main Street, in the early stages of the work, in sinking the foundations of the main piers. Pier No. 1 gave the most trouble. The unstable character of the soil on the south side of the stream was responsible for this. Extra heavy timber sheet-piling and a large number of heavy shores of timber and steel were used. In order to give the material on the hillside south of the pier the least possible chance to slide, the excavation was made in four sections. The first section excavated was the down-stream quarter of the pier. This excavation was about 20 ft. in length, measured up and down stream. After rock was reached, the surface was carefully cleaned off and several drill holes were put down to test the rock. The surface of the rock was found to be very rough, with a slight drop in elevation away from the stream. The drill holes showed that the layers of rock were very thick and had only minute seams between them. There had been some fear that the pressure of the bank behind the pier might cause the pier to slide on the rock, before sufficient weight could be added to prevent it. With this in mind, preparations were made to cut a dowel in the rock so as to bond the pier and the rock together. The fear of sliding was dispelled when the extremely rough character of the rock was seen, and the dowel scheme was abandoned. The pier section was concreted up to the top of the 13 ft. 6-in. footing course, and the next section, which was the third quarter from the down-stream end, was excavated and concreted to the same height as the first. The up-stream quarter was next finished, and lastly the second quarter from the down-stream end. Of course, all longitudinal reinforcing bars in the portion of the pier below the top of the footing course, had to be spliced, on account of the method of construction adopted. Care was taken to bond the sections of the footing course together by vertical re-entrant joints extending the full height of the 13 ft. 6-in. course. Above the top of this course the pier was built as a monolith in the usual manner. In concreting the footing sections it was not thought advisable to remove all the shores as the concrete was carried up. Consequently, some of the steel members from the

old bridge, which had been used for shoring, were built into the pier and left.

At Pier No. 2 the only difficulty experienced was in driving the oak piles. These were about 20 feet long, and were driven to refusal by using a drop-hammer. Driving through the compact mixture of sand and gravel at this point was difficult. A bearing test of the material in the bottom of the pit after the piles were driven gave very little settlement under a load of about 8 000 lb. per sq. ft., with a loaded area of about 4 sq. ft.

At West Seventh Street there was considerable trouble with the erection of the structural reinforcement of the arch span. The equipment was not as good as that used at Main Street, and the work progressed slowly. The engineers had expected that this steelwork would be handled by derricks on the piers in much the same manner as at Main Street. The sub-contractor felt that the size of the job did not warrant this equipment, and decided to crib up the steelwork on falsework in the bed of the stream. This was finally accomplished, and the falsework was removed, after the field splices in the ribs had been riveted. While the riveting of the transverse members was under way, the erector removed some of the lateral rods in order to get at the rivets with less trouble. This was done in spite of the protest of the engineers' representative. The stiffness of the lateral connections held the arch in place for a while, but finally a strong wind, aided to some extent by the movements of the men at work on the span, caused the ribs to revolve sideways on the hinges, and then drop into the stream a short distance below. Fortunately, no one was killed, and but slight damage was done to the steelwork. The work of re-erecting and repairing the steel was begun, the ribs being supported on timber falsework in the stream in much the same manner as before. When the work was only partly finished, the freshet previously mentioned, which occurred in September, 1913, came down and washed out the falsework, again dropping the steel into the river. The work was finally replaced in good shape, after much loss of time and money. This freshet also did some damage to the falsework for the traveler at Main Street. Had the arches been under construction on falsework in the usual manner, much more serious damage would undoubtedly have resulted.

FIG. 20.—MAIN STREET VIADUCT.

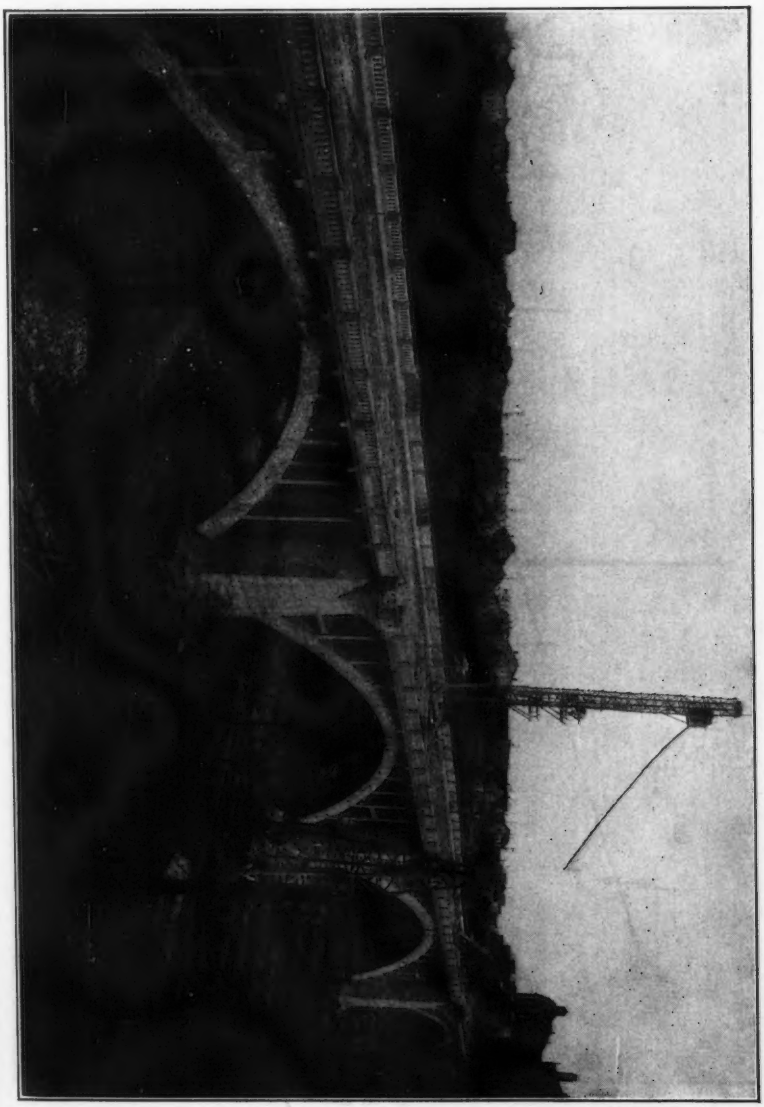

FIG. 20.—MAIN STREET VIADUCT.

FIG. 21.—WEST SEVENTH STREET VIADUCT.

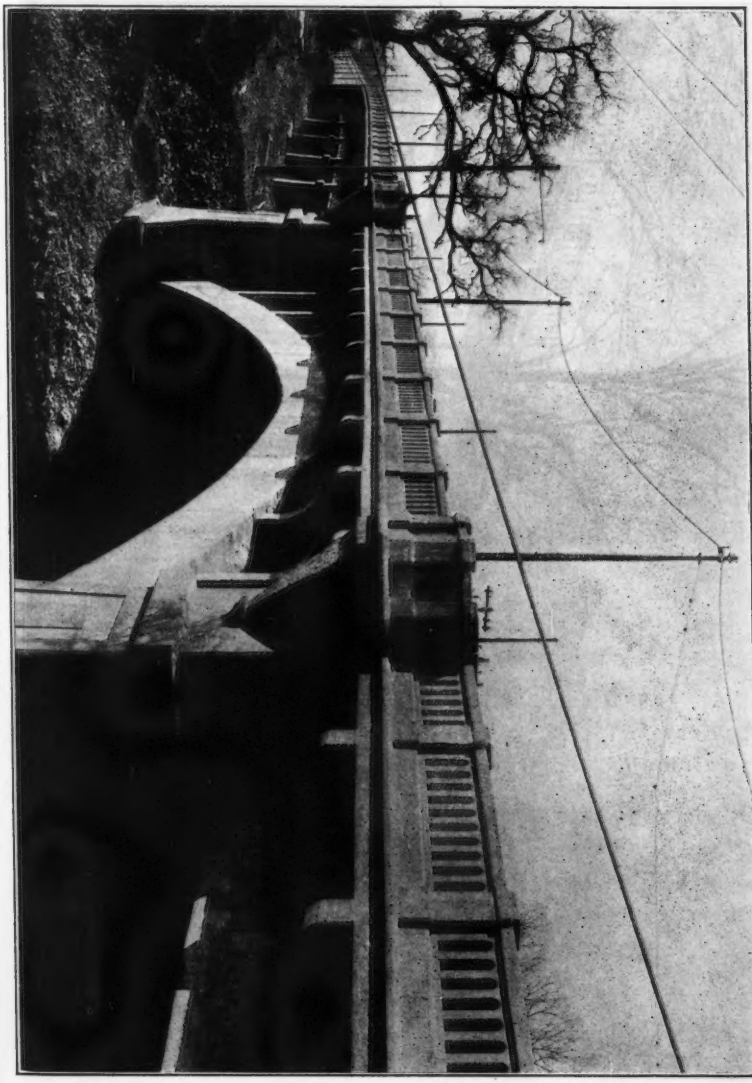

FIG. 21.—WEST SEVENTH STREET VIADUCT.

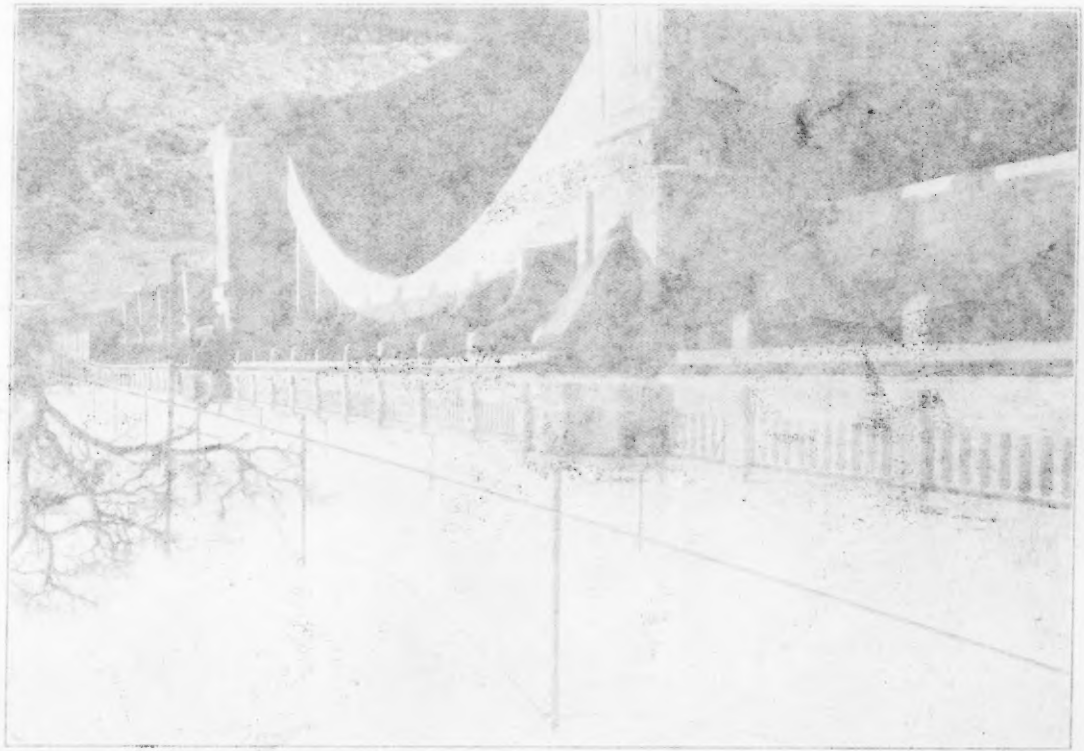

FIG. 22.—SAMUELS AVENUE VIADUCT.

FIG. 22.—SAMUELS AVENUE VIADUCT.

Figs. 12 to 22 show the work at various stages, and are self-explanatory.

Cost of Viaducts.—Tables 1 to 5 give the quantities for each of the viaducts, and the unit prices bid by the successful contractor. They also give the cost per linear foot of each structure, the cost per square foot of horizontal and of vertical projection, and the cost per cubic foot. The cost per square foot of vertical projection is based on the area of projection of each structure between the finished

TABLE 1.—COST OF MAIN STREET VIADUCT.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	4 720 cu. yd.	$0.35	$1 652.00	0.43
2	Foundation excavation	15 227 " "	1.00	15 227.00	3.96
3	Rock excavation	444 " "	2.00	888.00	0.23
4	(*a*) Concrete No. 1 (1:2:4)	10 611 " "	10.25	108 762.75	28.16
	(*b*) Concrete No. 2 (1:2½:5)	14 880 " "	6.85	101 928.00	26.40
	(*c*) Concrete No. 3 (1:3:6)	438 " "	6.25	2 737.50	0.71
5	Railings	3 875 lin. ft.	2.00	7 750.00	2.01
6	Structural steelwork	1 537 400 lb.	0.05	76 870.00	19.90
7	Steel reinforcing bars	1 375 150 "	0.035	48 130.25	12.46
8	Steel castings	205 460 "	0.07	14 382.20	3.72
9	Iron castings	11 173 "	0.04	446.92	0.11
10	Anchor-bolts and T. P. casings	21 311 "	0.06	1 278.66	0.33
11	Steel dowels	98	3.00	294.00	0.08
13	Rip-rap	836 cu. yd.	1.50	1 254.00	0.32
14	Manholes	2	50.00	100.00	0.03
15	Removing old bridge	.....	.....	2 500.00	0.65
17	Timber piles	194	10.00	1 940.00	0.50
	Total			$386 141.28	100.00

TABLE 2.—COST OF WEST SEVENTH STREET VIADUCT.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	5 424 cu. yd.	$0.25	$1 356.00	1.22
2	Foundation excavation	5 570 " "	0.61	3 397.70	3.07
3	Rock excavation	132 " "	3.00	396.00	0.36
4	(*a*) Concrete No. 1 (1:2:4)	2 890.4 " "	11.82	34 164.53	30.90
	(*b*) Concrete No. 2 (1:2½:5)	5 138 " "	7.25	37 250.50	33.69
	(*c*) Concrete No. 3 (1:3:6)	440 " "	6.15	2 706.00	2.45
5	Railings	2 120 lin. ft.	2.25	4 770.00	4.31
6	Structural steelwork	196 975 lb.	0.0475	9 356.31	8.46
7	Steel reinforcing bars	389 089 "	0.029	11 283.58	10.21
8	Steel castings	45 628 "	0.058	2 646.42	2.39
9	Iron castings	4 470 "	0.03	134.10	0.12
10	Anchor-bolts and T. P. casings	7 063 "	0.047	331.96	0.30
11	Steel dowels	30	1.50	45.00	0.04
12	Sidewalks on earth fill	5 240 sq. ft.	0.21	1 100.40	1.00
13	Rip-rap	363 cu. yd.	2.00	726.00	0.66
14	Manholes	4	38.00	152.00	0.14
15	Removing old bridge	.....	.....	750.00	0.68
	Total			$110 566.50	100.00

TABLE 3.—Cost of Samuels Avenue Viaduct.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	1 111 cu. yd.	$0.30	$333.30	0.58
2	Foundation excavation	2 739.6 " "	1.00	2 739.60	4.78
3	(*a*) Concrete No. 1 (1 : 2 : 4)	1 237.2 " "	12.00	14 846.40	25.91
	(*b*) Concrete No. 2 (1 : 2½ : 5)	3 413.6 " "	7.50	25 602.00	44.68
4	Railings	906 lin. ft.	2.00	1 812.00	3.16
5	Structural steelwork	12 739 lb.	0.05	636.95	1.11
6	Steel reinforcing bars	247 755 "	0.03	7 432.65	12.97
7	Iron castings	3 420 "	0.04	136.80	0.24
8	Anchor-bolts and T. P. casings.	6 388 "	0.06	383.28	0.67
9	Rip-rap	690.25 cu. yd.	2.00	1 380.50	2.41
10	Timber piles	200	10.00	2 000.00	3.49
	Total			$57 303.48	100.00

TABLE 4.—Cost of East Fourth Street Viaduct.

Item No.	Description.	Quantity.	Unit price.	Cost.	Percentage.
1	Grading	4 107 cu. yd.	$0.40	$1 642.80	3.11
2	Foundation excavation	2 832 " "	1.00	2 832.00	5.37
3	(*a*) Concrete No. 1 (1 : 2 : 4)	1 236 " "	12.00	14 832.00	28.11
	(*b*) Concrete No. 2 (1 : 2½ : 5)	2 734.5 " "	7.50	20 508.75	38.87
4	Railings	905 lin. ft.	2.00	1 810.00	3.43
5	Structural steelwork	12 739 lb.	0.05	636.95	1.20
6	Steel reinforcing bars	238 061 "	0.03	7 141.83	13.54
7	Iron castings	5 420 "	0.04	136.80	0.26
8	Anchor-bolts and T. P. casings.	6 388 "	0.06	383.28	0.73
9	Rip-rap	907 cu. yd.	2.00	1 814.00	3.44
10	Timber piles	82	10.00	820.00	1.56
11	Removing existing piers	.....	.....	200.00	0.38
	Total			$52 758.41	100.00

TABLE 5.—Summary of Cost of Four Viaducts.

	Main Street.	West Seventh Street.	Samuels Avenue.	East Fourth Street.
Cost per linear foot of structure	$244.60	$123.45	$141.29	$130.59
Cost per square foot of horizontal projection	3.66	2.10	3.28	3.02
Cost per square foot of vertical projection	6.34	6.93	3.53	3.60
Cost per cubic foot of volume between finished ground line and crown of roadway	0.091	0.119	0.082	0.083
Total estimated cost, including paving, lighting, and engineers' fees	$428 882.00	$127 472.00	$63 584.00	$58 766.00

ground line and the top of the roadway. For the horizontal projection, the extreme width over the copings or stringer mouldings was taken. In computing the cost per cubic foot, the volume included between vertical planes through the copings, and between the finished ground line and the top of the roadway, was taken. The total, on which the above unit costs are based, is the cost of the structure based on the contractor's unit prices and the quantities as given in the tables, to which has been added the cost of the paving, lighting, and engineers' fees.

Personnel.—The work was designed and its execution supervised by Messrs. Brenneke and Fay, Consulting Engineers, of St. Louis, Mo. The writer, as Principal Assistant Engineer for that firm, was responsible for the design of the viaducts, and exercised executive supervision over the work. L. H. Faidley, Jun. Am. Soc. C. E., had charge of the detailed work of preparing the plans, and William Holden, Jun. Am. Soc. C. E., was in direct charge as Resident Engineer.

The contract for the Main Street Viaduct was awarded to the Hannan-Hickey Brothers Construction Company, of St. Louis. The Tarrant Construction Company, of Fort Worth, had the contract for the West Seventh Street Viaduct, and the other two structures were awarded to the McKensie-Williams Construction Company, of Webb City, Mo. The Virginia Bridge and Iron Company, of Roanoke, Va., were the sub-contractors for the structural steelwork of the Main and West Seventh Street structures, and erected the steelwork at Main Street. Austin Brothers, of Dallas, Tex., were the sub-contractors for the erection of the steelwork at West Seventh Street.

In conclusion, the writer wishes to express his thanks to Mr. Faidley, and to Mr. Holden for valuable assistance in the preparation of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ADDRESS AT THE ANNUAL CONVENTION, IN BALTIMORE, MD., JUNE 2D, 1914.

By Hunter McDonald, President, Am. Soc. C. E.

In accordance with the provisions of the Constitution, the President must deliver an Annual Address. It is not difficult to understand why such a provision was placed in the Constitution of earlier days. The publications of the Society were then the principal means for disseminating information on engineering subjects and exchanging views thereon. In the present day, however, each separate branch of the Engineering Profession is provided with its own special organ, published for profit, and they vie with each other in presenting accounts of engineering work in most attractive and readable form. Society journals have become mere repositories of facts and opinions, and are not often read voluntarily, but are usually carefully preserved and are invaluable as sources of reference by reason of their authentic character and prestige. We should carefully guard and preserve the high quality of our publications, and all papers intended to promote private interests commercially should be denied admission.

In the earlier days the President was charged with the duty of reviewing the engineering progress of the previous year. A literal compliance with such a provision now would be a stupendous task, and, even in the hands of a skilled and entertaining author, would be uninteresting to the members because they have seen it all in such very attractive form from week to week in engineering journals.

Fortunately for me, and doubly fortunate for my audience, the character of the presidential address is no longer prescribed, and any amount of latitude seems permissible; at least such seems to be a reasonable inference from a cursory examination of some of the addresses of my predecessors in office.

Taking advantage of such liberty, and having due regard for the patience of my audience, I shall briefly refer to some of the more important matters of interest which the present year brings to engineers, and shall then direct attention to some matters concerning the welfare of our Society and the profession of which it is the foremost exponent.

It is a matter of pride and cause for congratulation that the most stupendous work of civilization, the Panama Canal, is about to be presented to our Government by men of our profession, completed greatly in advance of the time when it was due, and absolutely free from any taint of scandal, graft, or incompetency in any of the stages of its progress.

Fresh from the glories of this achievement, our Government has entered upon two more gigantic tasks to be performed in untried fields. These are the valuation of the railroad and telegraph properties of the United States, and the building of railroads in Alaska. The first of these projects is engaging the thought and time of a large number of our profession, and has already resulted in greatly increasing the demand for and compensation of engineers at a time when, on account of general stagnation in industrial development, many would otherwise be idle. In this work engineers have the opportunity to show to the country that they, more than any politicians or lawyers, are qualified to ascertain and fix the underlying principles upon which this valuation should be made, and then to follow this up by the actual working out of the results. It is a matter of encouragement to our brethren—although probably not to those of our citizens at large, who have given the matter much thought—to reflect, that from five to ten years will be required to complete the task, and, when it is done, a large force of engineers must be constantly employed, both by the railways and the Government, to keep the work up to date. It is not for us to say whether or not "the game is worth the candle", but to address ourselves to the task set before us and demonstrate our fitness for the responsibility.

The science of such valuation is new. In 1897 it became my duty to serve as one of a committee to value a piece of railroad property for a joint terminal. Being inexperienced, I sought from the pages and editors of technical journals and from other engineers some information as to the basis for such valuation. I was unable at that time to secure it. Now there is so much in print on the subject that is with difficulty that the chaff can be separated from the grain. We have a Special Committee which is wrestling with this problem and its preliminary reports indicate that important results are to be expected from its labors.

A commission of engineers is now charged with the duty of selecting a route for the main line of a railroad from some harbor on the southern coast to the central part of Alaska, and to determine what and where branches are to be built. When the location is determined, it is probable that the building of the roads will be entrusted to the United States Corps of Engineers. It seems improbable that the roads will ever be operated by any agency other than the National Government. An opportunity presents itself here for civil engineers to demonstrate their fitness for responsibilities other than the mere application of mathematics. A civil engineer is now Governor of the Canal Zone, and another should be General Manager of the Alaskan Railroad when the time for its operation arrives. It is not necessary to state that his staff should be composed of engineers. An engineer has recently been chosen as manager of one of our large cities. Many are being placed on City Commissions.

Persons who are charged with the responsibility for the wise expenditure of public money are more and more inclined to turn to the engineer for advice as to how to do it, and to recognize the fact that civil engineers are fit for something else than preparing blue prints, setting stakes, and running instruments. We are now on the threshold of new responsibilities, and we must set ourselves faithfully to the task of preparing to meet them. Good engineering consists largely in meeting emergencies with the resources at one's disposal. The man who does it is always in demand.

Much of the space in the addresses of some of our Past-Presidents has been devoted to discussing the status of the engineer, the ethics of the profession, and how to compel an apparently unwilling public to recognize the fact that engineering is a profession. Complaint is

rife that we do not get our just deserts, and that something not clearly defined must be done to remedy this injustice.

I have been tempted to delve somewhat deeper into the origin and present meaning of the word "engineer" than my predecessors in office have done. Realizing my own lack of fitness for the task, as far as the origin is concerned, I called in a specialist on etymology, Dr. Bert E. Young, Professor of Romance Languages of Vanderbilt University, and lately honored by the French Government with the decoration of Officer of Public Instruction for distinguished services in the promotion of French literature.

I quote his statement in full:

"I take pleasure in furnishing the best information I have as to the origin of the word 'engineer':

"*Engineer,* formerly *enginer,* or sometimes *ingener,* Middle English *engyneour,* from the Old French *engignier* or *engigneour,* or shorter *engineur.* This from Middle Latin *ingeniarius,* one who makes or uses an engine, especially a war engine. *Engine* is from Latin *ingenium,* an invention, an engine.

"The French word *ingenieur,* a semi-learned form, seems to come down by a slightly different road, being from the hypothetical Middle Latin word *ingeniator* from *ingeniare* from *ingenium,* an engine. Or else the Modern French *ingenieur* is an alteration of the better form *engigneur,* mentioned in first paragraph.

"*Genie* from Lat. genius, orig. a spirit, extended as a term of war to the sense of *art* of attacking or defending fortified places, etc., and then to the arm of the service devoted to military or naval engineering.

"*Ingenuity* from French *ingenuite* from Lat. *ingenuita (t)s* from *ingenuus,* native, born of free parents, frank, generous. *Ingenious* from Lat. *ingeniosus,* gifted with genius or natural capacity. Lat. *ingenium* from Old Latin *genere.*

"Apparently all go back to the ubiquitous root, $\sqrt{\text{gen}}$, to bear, to produce, to give birth to.

"Misc. remarks:

"Considering the derivation of ingenuity, there must have been confusion of Lat. *ingenuus* and Lat. *ingeniosus.* These should be almost opposite in meaning. I suppose an *engineer* ought to be both *ingenious* and *ingenuous,* artful and artless, sophisticated and unsophisticated, bond and free. According to the above, I should say that the man who pronounces *injine* is only atavistic.

"Ex.—'In hys court was a false traytoure, that was a grete Yngynore (plotter). (Halliwell, 1420).'

"Ex.—'The most prime Engineers of Oaths that ever the world knew. (Butler, 1680)'.

"Ex.—'For 'tis the sport to have the enginer hoist with his own petar. (Shak., 1602, Ham. iii, 4, 206).'

"Ex.—'The Ingener of this practice—was a Portugal. (Carew, 1620).'"

From the Oxford Dictionary, which no doubt gives the prevailing English view, I have extracted the following definitions and examples of the use of the word "Engineer".

"One who contrives, designs or invents. Ex. 1611, Rich, Honest age, 1844 (36) 'Yet you cannot deny them to be the devil's engineers'.

"1635, R. Gibbs in Spurgeon 'That great engineer Satan'.

"A constructor of military engines (Obsolete). One who designs and constructs military works for attack or defense. 1325 Coer De Leon 1387—'a tour ful strong, that queyntyly engynours made'.

"A soldier belonging to the division of the army called engineers, composed of men trained in engineering work. Ex., Porter's History, Royal Engineers. 'This day (May 26th, 1716) may thereafter be taken as the one on which the engineer branch blossomed into a distinct corps.'

"One whose profession is the designing and constructing of works of public utility, such as bridges, roads, canals, railways, harbors, drainage works, gas and water works. From the 18th Century, also civil engineers (from the military).

"Ex. 1792 Smeaton Reports (1797) I pref. 7. 'The first meeting of this new constitution, the Society of Civil Engineers, was held on the 15th of April 1793.' Ex. Eddyston L. introduction—'My profession of a civil engineer.'

"A contriver or maker of engines (specifically mechanical engineer).

"One who has charge of a steam engine. In England only with reference to marine engines. In the United States often applied to the driver of a locomotive. Ex. 1860, Bartlett's Dictionary. American, Engineering, 'the engine driver on our railroads is thus grandiloquently designated'.

"Engineman, one who works or helps to work a fire engine. One who attends to a stationary engine. The driver of a locomotive."

From other English authorities:

March's Thesaurus Dictionary. "One versed in any branch of engineering. To manage skillfully. The science and art of building, making and using engines and machines."

Hallowell's archaic and provincial words. "Engin, wit, contrivance (Lat.). Inginous, inventive (Jonson)."

From Joseph Wright's English Dialect Dictionary. "In 1649 Gray wrote 'Master Beaumont a gentleman of great ingenuity and rare parts adventured into our mines, who brought with him many rare engins. The memory of these 'rare engins' survives in the name of the seam

which he appears to have discovered, still called the engin seam." It will be noted that Beaumont was practicing civil engineering at quite an early date.

From the Century Dictionary, presenting the American view, the following:

"A person skilled in the principles and practice of any department of engineering. Engineers are classified according to the particular business pursued by them, as *military, naval* or *marine, civil, mining, mechanical* and *electrical* engineers.

"To work upon; ply; try some scheme or plan upon.

"Ex.—'Unless we *engineered* him with question after question, we could get nothing out of him.'—*Cowper.*

"To guide or manage by ingenuity or tact; conduct through or over obstacles by contrivance or effort; as to *engineer* a bill through Congress."

Through the kindness of our Secretary, Mr. Hunt, I have been able to secure the following translations from the Dictionnaire de la Langue Française:

"One who plans, lays out and directs field work and fortifications, for attacking, defending or fortifying places.

"Ex.—'Then he (Peter the Great) has had even a school of engineers in a country where no one (before him) knew the elements of geometry, Volt. *Charles XII,* I.'

"One who directs [private] works or public works, such as the construction and maintenance of roads and bridges, working of mines, etc. Engineer of bridges and roads. Mining Engineer, Naval Engineer or Marine Engineer, Railroad Engineer.

"Civil Engineer, name given to engineers who are not trained in technical schools or who work for private industry.

"Naval Architect or Engineer, one who applies himself to the art of naval construction.

"Topographical Engineer, one who draws maps. Hydrographical Engineer, See Hydrography.

"Engineer for mathematical instruments, one who makes mathematical instruments."

Also, from Cotgrave's Dictionary of the French and English tongue:

"Enginier —An engineer; a maker of engins.
Enginer —Engigner.
Engigner —A deceiver, beguiler, cousener; also an enchanter.
Engin —An engin, toole, instrument; also understanding, policie, reach of wit; also suttletie, fraud, craft, wiliness, deceit.
Ingenieur—An enginer, engine-maker; fortifier."

Mr. Hunt also furnished me the following translation from Grimm's Deutsches Wörterbuch:

"Engineer, n.—This word of the present day, for military architect, surveyor, introduced from a foreign language in the 17th century represented an artful calculating person in general. Ex.—'He who wishes to bring Germans into an understanding must be a very good and wise engineer (schemer, calculator)'. Weidner's, Zinkgres, 3, 158."

No trace of the use of the word "Engineer" or of any of the words from which it is derived in describing the building of any of the ancient structures has been found. That engineers of great skill existed is evidenced by monumental and written records, but they were designated otherwise, as for example: Frontinus, in his water supply of the City of Rome, as translated by Clemens Herschel, M. Am. Soc. C. E., mentions *moduli* (levelers), *scientia* (builders), *agrimensores* and *geometricii* (land surveyors), *architectos* (architects), and *Curator Aquarum* (Commissioner of Water Works), but none of the Latin words from which the word "engineer" is derived, are used in connection with the process of building or maintaining. The conclusion is inevitable that the name at least is comparatively modern, and that the attempt to class the calling among the learned professions is still more so.

John Smeaton was the first engineer to style himself Civil Engineer (probably in 1782).*

In the Annual Address of Mr. Charles Hawksley, President of the Institution of Civil Engineers, 1901, an extensive quotation is made from the Report of the Council, for the Session, 1885-86. A portion only is repeated here:

"'The honor of having originated the Institution is often assigned to Smeaton, or to Telford, but the idea is erroneous in both cases. Smeaton died many years before it was thought of, and Telford only joined it after its establishment.'

"A Society of Engineers, still existing, was founded by Smeaton in 1771; it includes many of the most eminent members of the profession, but it is rather of the nature of a social club than of a scientific association, and has no connection with this Institution."

To those who are interested in the beginnings of engineering, I would recommend an examination of the two papers above referred

*See discussion by Clemens Herschel, M. Am. Soc. C. E., of Paper No. 472, entitled "Beginnings of Engineering", by the late J. E. Watkins, Assoc. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXIV, p. 382.

to and the following: Annual addresses of Past-Presidents William E. Worthen, 1887 (*Transactions,* Vol. XVII), and William P. Shinn, 1890 (*Transactions,* Vol. XXII), also of Charles B. Vignoles, F. R. S., Past-President, Institution of Civil Engineers, 1870 (*Minutes of Proceedings,* Vol. XXIX).

On page 604 of *Engineering News,* May 26th, 1910, is an entertaining paper entitled the "Origin and History of the Profession of Civil Engineering". A foot-note describing it reads as follows:

"An appendix to the Presidential address of Jas. C. Inglis, President of the Institution of Civil Engineers of Great Britain. Abridged from the account contained in the life of Sir William Fairbairn by the late Dr. William Pole, F. R. S., Hon. Secretary of the Institution."

A portion of the beginning is quoted:

"The root of both the words, Engineer and Engine, is found in the Sanscrit *jan,* to be born, from which came the Greek form γεν, and the Latin *gen,* the latter being embodied in the old verb *genere,* with its compound *ingenere* (changed into *ingignere*), to implant by birth, and in their later substantive *ingenium,* an innate or natural quality.

"The old Latin verbs, *genere* and *ingenere,* gave rise to the French form, also a verb, *s'ingenier.* This is of great antiquity, and from its comprehensive and useful meaning it has continued in use down to the present time, being found continually in modern French writings. The import of it has nothing whatever to do with engines or machines, but is purely psychological. It is given in Littre's great French dictionary:

"'*Chercher dans son genie, dans son esprit, quelque moyen pour reussir.*'

"Now, all authorities, including our own great Lexicographer, agree that this word is the true origin of the word 'Engineer', and thus we arrive at the interesting and certainly little-known fact that an Engineer is, according to the strict derivation of the term, not necessarily a person who has to do with engines, but any one who seeks in his mind; who sets his mental power in action, in order to discover or devise some means of succeeding in a difficult task he may have to perform."

There are sufficient divergences of authority and definitions in the above quotations to give room for any amount of argument. I will be content with simply recording them.

These matters are mentioned primarily as leading up to a consideration of the condition in which the profession in this country

now finds itself. There is ample evidence that the word "Engineer" is now applied alike to the designers of engineering works and to the operators of engines. It is also evident that civil engineering is now considered by many to be a branch of engineering. In the language of the late Grover Cleveland "It is a condition and not a theory which confronts us." We could not, if we desired, compel the public to call locomotive engineers "enginemen" or "engine drivers", nor stationary engineers, "power superintendents", notwithstanding the fact that such they are. President Worthen in the address above referred to says:

"I would drop the term civil as embracing too large a profession, and adopt prefixes, as military, hydraulic, railway, mechanical, steam, electric and other divisions, when these are adopted as specialties."

His idea seems to have been merely to divide civil engineers into different classes. He does not mention civil engineers as one of the classes.

Until recently, I have also been inclined to the view that we should drop the word "Civil" from the name of our Society, but a closer study of the matter confirms me in the opinion that we should retain it. I think there can be no question but that civil engineering embraces all engineering except that connected with the actual conduct of war. The fact that the military engineer "in piping times of peace" performs the work of the civil engineer constitutes no distinction.

Conscious of our pre-eminence, this Society has been content to sit still and let the branches of our profession grow up independently until they have become of such importance as to force us to accept the meaning of the word "Civil Engineer" as indicating a specialty or division of the profession. This is a false position, and we alone are responsible for it. Smeaton evidently intended to make it a specialty, but he could not have known, at the time he referred to it, how intricate the divisions of the specialty were to become. Had we pursued a course similar to that of the engineers of Germany we would not to-day find ourselves in the humiliating position of being asked to substitute the word "Professional" for the word "Civil" in the bill recently pending before the New York Legislature for the regulation of the practice of civil engineering, before the other societies would co-operate with us in securing a bill which would be acceptable to the entire profession.

Believing that it would be interesting to know the number of the various kinds of engineers which go to make up our membership, I requested the Secretary to prepare from the List of Members a classification. After much labor it has been accomplished and is presented herewith in tabular form. It will be noted that in this classification our Secretary treats civil engineers as one of the divisions or classes. Under the conditions, he could not do otherwise.

I have alluded to the status of the profession in Germany, and will now describe its condition more at length, also referring briefly to conditions that exist in a few other countries. This is done with a view of comparing conditions in these countries with those in our own.

My information regarding Germany is taken from an article in *Engineering,* London, December 20th, 1912, entitled "The Verein Deutscher Ingenieure". It was founded in 1856. Quoting verbatim, the dominating ideas are expressed under three heads:

"1. The society must in its activities embrace the whole of Germany. [This is remarkable in that it anticipated Bismarck's federation by 15 years.]

"2. Local sections (*Bezirksverein*) must be founded in all parts of Germany.

"3. A good journal must be published."

In 1865 it had 1 000 members; in 1875, 3 000; in 1885, 5 300; in 1895, more than 10 000; and in 1912, about 24 000.

It has for its object, as indicated by the printed statutes, the intimate co-operation of the intellectual forces of German technical science for the benefit of the entire industries of the Fatherland. This object is to be attained by meetings of the entire Verein and its local sections, the *Journal,* the establishment of libraries and reading rooms, the editing of important technical works, the instituting of engineering research on subjects of industrial importance. The membership embraces (*a*) engineers, and (*b*) those capable of advancing the objects of the Union. Contributions are made from the general fund toward the support of local sections. All members living within the Empire must belong to local sections. In one year 300 business meetings were held and 400 lectures on technical subjects were delivered. The question box is in use at all sections, and a general technical committee answers all questions not answered by the members of the section at the regular meeting. These local sections, as a rule, are in touch

with the educational and industrial powers of the neighborhood. Courses of instruction and excursions through works are arranged for, and visits are frequently exchanged between sections. "Sentiment plays a much more prominent part in the life of the local sections than English [and, I may say, American] prejudice would allow." Honors are bestowed on leading members as they reach advanced age. and "the social graces are not neglected." Headquarters, libraries, and reading rooms are often in hotels, to the mutual benefit of the section and the proprietor. Weekly meetings are held with tables reserved. Ladies attend the monthly meetings. Play on the skittle alley, and free and easy beer evenings are monthly features of some sections. The Board of Management of seven is selected by delegates from the sections, and this council controls, except on extraordinary occasions when they meet with the delegates. Local sections are self-governing, provided their constitutions are approved by the council and their actions further the society's objects. General annual meetings are held. The backbone of the organization is the *Journal*. In 1911 it had 2 200 pages of text, exclusive of the advertisements. News and proceedings of local sections are published. Books are reviewed with conscientiousness. Brief abstracts are given of important articles in more than 60 leading journals. They follow closely on the article itself, and are then reprinted on one side of the sheet, with room for marginal notes. Much space is devoted to correspondence. Original researches are encouraged, and grants are made from the general fund therefor. Medals are awarded for specially good work, and bad work is not permitted. In the supplement to the *Journal*, questions of financial, political, economical, and literary news are treated. The general aim of the supplement is to afford the engineer knowledge of the wider world which he serves, and to bring the needs of the engineering world before the sociologists, economists, and politicians. "The enormous number of members and the influential positions which many of them hold, make the Verein a power in the land, with a voice for the industrial elements such as no other country possesses." It has had great influence on technical education, and has banished Latin and Greek from the curriculum.

A new movement to inject engineering ideas into municipal and State administrations has been inaugurated and is meeting with great success. The framing of patent laws and the purification of

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Character of Employment of the Membership of The American Society of Civil Engineers.

In the List of Members, February 10th, 1914, a large percentage of the total membership (19.72) do not give any specific occupation, and therefore cannot be classed. The numbers and percentages in the following Table, therefore, are based on the 5 864 whose occupation is given in such a way as to enable them to be classed at least with some degree of probable accuracy.

		Character of Employment.						Private Practice.												
		Railroads.	National Governments.	States and Counties.	Municipalities.	Private Companies and Individuals.	Technical Schools.	General.	Mechanical.	Electrical.	Sanitary.	Hydraulic.	Structural.	Mining.	Chemical and Metallurgical.	Highway.	Heating and Ventilating.	Miscellaneous.	Total.	Percentage.
Hon. Members.	Consulting Engineers	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	33.3
	Civil Engineers	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	33.3
	Corps of Engineers	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	33.3
	Total	1	1	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	3	.....
	Percentage	33.3	33.3	.....	.....	33.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	100
Cor. Members.	Consulting Engineers	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	.....
	Total	.....	.....	.....	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	.....
	Percentage	.....	.....	.....	.....	.....	.....	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	100
Members.	Consulting Engineers	5	3	4	11	18	.....	545	10	1	15	59	17	19	3	2	2	19	733	27.0
	Civil Engineers	2	26	.....	.....	7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	35	1.3
	Chief Engineers	102	14	54	124	125	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	419	15.6
	Assistant Engineers	200	163	61	163	183	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	770	28.5
	Instrumentmen, etc	1	4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	5	0.2
	Presidents, Managers, Superintendents, and Other Officials	72	12	5	19	284	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	392	14.5
	Professors and Associate Professors	.....	.....	.....	.....	.....	110	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	110	4.1
	Assistant Professors and Instructors	.....	.....	.....	.....	.....	6	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	6	0.2
	Contracting Engineers	.....	.....	.....	.....	11	.....	92	.....	.....	.....	.....	11	.....	.....	.....	.....	2	116	4.3
	Corps of Engineers	.....	55	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	55	2.0
	Architects	1	1	.....	.....	.....	.....	36	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	39	1.4
	Commercial Representatives	.....	.....	.....	.....	15	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	15	0.6
	Miscellaneous	1	2	.....	1	4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	8	0.3
	Total	384	280	124	318	647	116	673	10	1	15	59	28	19	3	2	2	22	2 703	.....
	Percentage	14.2	10.3	4.6	11.8	23.9	4.3	24.9	0.4	0	0.6	2.2	1.0	0.7	0.1	0.1	0.1	0.8	.....	100

RECAPITULATION.

Total Membership.	Character of Employment.					Private Practice.										
	Railroads.	National Governments.	States and Counties.	Municipalities.	Private Companies and Individuals.	Technical Schools.	General.	Mechanical.	Electrical.	Sanitary.	Hydraulic.	Structural.	Mining.	Chemical and Metallurgical.	Highway.	Heating and Ventilating.	Miscellaneous.	Total.	Percentage.
Consulting Engineers	7	3	6	12	28	.....	790	13	3	28	99	36	32	10	5	2	35	1 109	18.9
Civil Engineers	9	33	1	.....	16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	59	0.9
Chief Engineers	33	33	100	210	252	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	716	12.2
Assistant Engineers	125	311	271	456	703	2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2 882	49.8
Instrumentmen, etc.	496	65	14	18	39	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	158	2.7
Presidents, Managers, Superintendents, and Other Officials	97	39	13	38	613	3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	798	13.6
Professors and Associate Professors	.....	.....	.....	.....	.....	188	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	188	3.2
Assistant Professors and Instructors	.....	.....	.....	.....	.....	77	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	77	1.3
Contracting Engineers	.....	.....	.....	.....	30	.....	191	.....	.....	.....	.....	15	.....	.....	.....	.....	5	241	4.1
Corps of Engineers	.....	70	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	70	1.2
Other Military Engineers	.....	5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	6	0.1
Architects	1	1	.....	2	.....	.....	51	.....	.....	.....	.....	.....	2	.....	.....	.....	.....	56	0.9
Commercial Representatives	.....	.....	.....	.....	38	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38	0.7
Miscellaneous	3	4	.....	1	14	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2	24	0.4
Total	752	587	411	732	1 793	271	1 032	13	3	28	99	51	32	10	5	2	43	5 864	.....
Percentage	12.8	10.0	7.0	12.5	30.6	4.6	17.6	0.2	0.1	0.5	1.7	0.9	0.5	0.2	0.1	0	0.7	.....	100

CHARACTER OF EMPLOYMENT OF THE MEMBERSHIP OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.—(*Continued.*)

	Character of Employment.					Private Practice.											
	Railroads.	National Governments.	States and Counties.	Municipalities.	Private Companies and Individuals.	Technical Schools.	General.	Mechanical.	Electrical.	Sanitary.	Hydraulic.	Structural.	Mining.	Chemical and Metallurgical.	Highway.	Heating and Ventilating.	Miscellaneous.	Total.	Percentage.
Juniors.																			
Consulting Engineers			1		1		12			2	2	1	1				1	20	3.4
Civil Engineers		8	8	13	17													4	0.7
Chief Engineers				56	165	1												38	6.4
Assistant Engineers	48	20	48	11	28	1												338	57.5
Instrumentmen, etc.	14	29	14															97	16.5
Presidents, Managers, Superintendents, and Other Officials	1				30	10												31	5.2
Professors and Associate Professors						24												10	1.7
Assistant Professors and Instructors						3	11											24	4.1
Contracting Engineers		4																14	2.4
Corps of Engineers																		4	0.7
Other Military Engineers		3			3													3	0.5
Commercial Representatives					1													3	0.5
Miscellaneous	1																	2	0.4
Total	64	59	71	80	245	39	23			2	2	1	1				1	588	
Percentage	10.9	10.0	12.1	13.6	41.7	6.6	3.9			0.3	0.3	0.2	0.2				0.2		100
Fellows.																			
Presidents, Managers, Superintendents, and Other Officials	1				10											1		11	91.7
Miscellaneous																		1	8.3
Total	1				10											1		12	
Percentage	8.3				83.4											8.3			100

technical language has received attention. It has fixed mechanical standards as well as fees for engineers and architects. Needy members are assisted with charity. No correspondence tuition is allowed to be advertised in the journal. Branches in foreign countries are contemplated.

I have dwelt at considerable length on the abstract of this article because it shows what might have been accomplished in this country had the management of our Society shown the same interest in the profession at large as it has in the welfare of its members only.

In France the good of the entire profession has always been the object of the Société des Ingenieurs Civils, which is the title of the organization of the free engineers as distinguished from that of the engineers of the Ponts et Chaussées and other Government engineers. This Society was founded in 1848. It flourished from the beginning, and received many donations. In 1877 its new building was completed, and the Society moved into it. Government engineers are not eligible, but it was organized largely by Alumni of the Government schools. Its avowed objects are:

1. To throw light by discussion on obscure questions in civil engineering;
2. To assist in aiding the development of applied science auxiliary to civil engineering and to assist industries;
3. To extend professional teaching among workmen and shop foremen;
4. To investigate questions of industrial economy of administration, to increase the power production and wealth of the country;
5. To insure close relations among its members;
6. To act as a kind of employment bureau for its members; and
7. To establish a fund in case of necessity.

An examination of the year-book of the Society indicates a very prosperous condition, both as to finances and membership. The members are scattered over many colonies and foreign countries. Sections exist in all provinces.

The following is quoted from the List of Members of the Institution of Civil Engineers of Great Britain:

"The Institution of Civil Engineers was first established, and has since been incorporated by Royal Charter, for the General Advance-

ment of Mechanical Science, and more particularly for promoting the acquisition of that species of knowledge which constitutes the profession of a Civil Engineer; being the art of directing the great sources of power in Nature for the use and convenience of man, as the means of production and of traffic in States both for external and internal trade, as applied in the construction of roads, bridges, aqueducts, canals, river navigation and docks, for internal intercourse and exchange; and in the construction of ports, harbours, moles, breakwaters and lighthouses, and in the art of navigation by artificial power for the purposes of commerce; and in the construction and adoption of machinery; and in the drainage of cities and towns."

This organization is very similar to our own, and its status is so well known as not to require detailed reference here. All members admitted below the grade of Member are required to pass examinations. The list contains the names of more than 9 000 members of all grades.

The Canadian Society of Civil Engineers was organized in 1887. Its objects are:

1. To facilitate the acquirement and exchange of professional knowledge among its members; and
2. To encourage original investigation.

Article 2 of the By-Laws reads as follows:

"The term 'Civil Engineer', as used in this Society, shall mean all who are, or have been, engaged in the designing or constructing of railways, canals, harbours, lighthouses, bridges, roads, river improvements or other hydraulic works, sanitary, electrical, mining, mechanical, or military works, or in the study and practice of navigation by water or air, or in the directing of the great sources of power in nature for the use and convenience of man."

It will be noted that this Society conceives civil engineering as embracing all engineering activity.

The standard of membership is not as high as in the Institution or our Society, but candidates who are not graduates of recognized technical colleges are required to pass examinations. It is divided into four sections: Electrical, Mechanical, Mining, and General. Note the absence of Civil. Provision is made for branches in important centers. They have a published code of ethics, and in two Provinces, Manitoba and Quebec, the practice of engineering is limited by law to members of the Society. Efforts are in progress to secure similar

laws in all other Provinces. The list shows 2 627 members of all grades.

Members need no description of our own organization. While our growth has been phenomenal, and the standard of our membership is generally maintained at a high level, there is a wide-spread feeling that there is something wrong, and that we are not accomplishing the good there is in us. Section 3 of Article I of our Constitution reads as follows:

"Its object shall be the advancement of engineering knowledge and practice and the maintenance of a high professional standard among its members."

Our total membership in all grades is more than 7 400. I believe there are fully 40 000 men in the United States who are eligible for various grades of membership, in about the same proportion as the list is now made up. It is hardly safe to assume that the reason they are not members is because they cannot afford to be. I think it is safe to assume that though they value the prestige of membership, they feel that the benefits are too remote. The engineer's calling generally imposes isolation, and yet they all dearly love companionship and exchange of ideas with one another. What we need and ought to have is the establishment of State organizations and of sections in all important cities. This matter has been studied by a special committee and discussed at one of our Annual Meetings, but, because we felt that our duty ended with the welfare of our own members, as provided in the Constitution, the project met with no encouragement. It is true that we permit local members in large cities to band together and call themselves Associations of Members of the Am. Soc. C. E., but they, as organizations, have no voice in our government, cannot mingle with other local organizations, and, except in very large cities where no other sections already exist, the formation of prosperous associations is impossible. The history of such local associations is by no means flattering to our pride or good management. The result of our narrow policy has been that organizations, embracing members of all the callings allied to civil engineering, have sprung up in nearly all cities of 100 000 inhabitants, and other national engineering societies have their own local sections established in many of these centers, and are co-operating with other local organizations. Some of these societies are by no means local in scope. In the inde-

pendent local organizations the membership in many instances has not been of so high a standard as our own because of the necessity for a large membership in order to meet expenses, and, like the German and Canadian Societies, all classes are taken in. Nearly all these organizations contain some of our members who have joined, sometimes as a matter of civic pride and more often because they found at home that companionship which could not reach them through the medium of our *Transactions* from New York. All but a few of these local organizations are struggling for existence, and are unable to publish their papers and discussions. Some are prosperous, and one in Boston antedates our own organization by several years. It is the duty of this Society to extend a helping hand to all who need it, and to take the initiative in an effort to amalgamate gradually all engineering organizations into one homogeneous and co-operative mass, permitting each, as far as possible, to retain its name and identity. Some sacrifice on the part of all would no doubt be necessary. Union has not been possible among religious sects, but as I have heretofore shown, engineers are of necessity managers, and, with their spirit of fair play and good sense, ought to be able to get together. Call it what you will, the American Engineer Confederation, or better still, the American Confederation of Civil Engineers.

I would not lower our standard of requirement for membership, but, by extending local societies limited financial assistance where needed, we can raise the general standard, and, by doing this, can elevate the profession at large so as to make a printed code of ethics unnecessary.

The time has come for the engineer to possess his own. The public is awake to the advantages of sound engineering advice. We have it in our power, without taking sides in political contests, to exert a beneficial influence in National, State and Municipal affairs. Some of the matters in the conduct of which we should exert a marked influence are the following:

We should seek to stop the economic waste, caused by the improvement of unworthy projects for internal waterways and harbors, the means for which are often wrung from Congress on padded and duplicated commercial statistics, the principal object being to aid Congressmen in re-election.

We should seek to have the civilian engineers in the employ of the Government given proper recognition of their services in the matter of credit, rank, salary, and retired pay.

We should see that the mad schemes for flood control of our rivers are punctured before money is expended on them, and that practicable schemes, considering the entire drainage basin as a whole, are adopted and their execution begun on a sound basis as soon as possible.

In every State and in every city we should lend our organized influence toward the securing of safe and sane building laws, free from the bias of selfish private interests and trades unions; that public moneys derived from bonds and taxes are not squandered on impossible and unnecessary projects which have not been the subject of proper engineering scrutiny; that sanitary officers are chosen for their fitness, and not because they write M. D. after their names; and that sanitary methods of water supply and sewage disposal are adopted in the rural districts. We should make use of the non-technical press in all legitimate ways, in order to bring before the minds of the public at large the importance of the engineer in civil life. Whenever we let the people know that we are organized, not only for our own improvement and protection, but for the public good, we need have no fear of not receiving proper consideration. We are wont to twit our medical friends on the fact that they often bury their mistakes, while our's stand to plague us, but we dare not compare records with them on charities and public spirit.

Before undertaking to show others what they ought to do, we should first put our own house in order. Good engineering is good management. From our financial statement it would appear that we are in a very prosperous condition, which is no doubt true, and I wish to congratulate our past management on its wisdom and foresight in this respect, but are we getting all that we can out of our present resources?

Our Society house is located in what is fast becoming the business district of New York City. The present value of the land and building is not less than $600 000. Two large rooms are maintained as Auditorium and Assembly Room, respectively. One of these is used for the fortnightly meetings at which the attendance averages about 175. It will seat 500 persons. On the occasion of our Annual Meeting both are needed for the Smoker, but as most of the members prefer

the lower room, the 700 occupants find it uncomfortably crowded and badly ventilated. The annual ball was held this year at the Hotel Astor because the Society halls were inadequate.

There are 31 persons now employed in the building. It contains a library of 80 000 volumes, with room for 150 000, also a Reading Room which was used last year by 4 887 readers. Research work for resident and non-resident members is performed at cost by the Society's staff. Our stock of publications, costing about $25 000, is kept in the building.

Interest on the value of our property at 5% makes the rental we pay for this house about $30 000 per annum. It would seem possible and wise to issue bonds and erect a building, portions of which could be rented for a sufficient amount to pay our interest, establish a sinking fund, and give us rent free. The upper floors should be rented, preferably to engineer tenants, the street floors to merchants. The top part of the building would supply our offices, auditoriums, and perhaps a roof garden. The architectural possibilities should atone for any fancied loss of dignity. Using elevators nowadays is not considered a hardship. The amount now devoted to rent would go very far in aiding struggling local sections.

Pending the working out of these larger problems, there are minor steps which can be taken at once, which will add to the value of our membership.

The volume now called "Constitution and List of Members" should be published in the form of a year-book, containing all that it now contains, and, in addition thereto, a codification of the by-laws, which have been passed from time to time by the Board of Direction and are subject to change by it. There are other matters of general interest which might be profitably included.

The minutes of our Society are now published in the *Proceedings*. Few of the members preserve the *Proceedings* because most of the papers therein are duplicated in the *Transactions*. More important items of the *Proceedings*, including the Minutes of the Board of Direction and Annual Meetings and Conventions, should be included in the year-book, and would thereby be preserved without compelling the expense of binding and preserving volumes of *Proceedings*.

The classification which the Secretary has inaugurated should be a regular annual feature, and each member should be required to

return his calling at the same time as he does his address. Steps should be taken to provide automatically for transfer to the Associate class of all members who abandon engineering practice and devote themselves exclusively to the promotion or sale of patented articles and appliances.

The provisions for expelling members guilty of improper conduct should be made less complicated and the power be placed in the hands of the Board of Direction. The Board of Direction should be empowered, and required, to espouse the cause of any member who is the object of unjust attack. Fees should be paid to members for creditable extracts of engineering articles appearing in technical papers, and for unbiased book reviews, and the advisability of continuing the monthly notes of important engineering articles, now published in the *Proceedings,* should be investigated. There are many things in the "Constitution" that should be placed in the by-laws.

I am not prepared to suggest methods by which the ideals which I have heretofore rather vaguely outlined may be brought into realization, but I feel that whenever a thing ought to be done, a way can be found to do it, and I believe a representative committee of our Society should be appointed to study what we ought to do and how to accomplish it.

Before closing I would remind you that our next Annual Convention will probably be held on the Pacific Coast, and that, in connection with the Panama-Pacific Exposition, this Society is co-operating with four National Engineering organizations in the holding of an International Engineering Congress at this Exposition. The organization and conduct of this Congress is in the hands of a Committee on Participation, composed of the Presidents and Secretaries of each of these Societies and a local Committee.

It is expected that many foreign engineers will visit the Exposition and attend the Congress. They will come as individuals, in groups, and in organizations. A Special Committee on entertainment of these foreigners has been appointed by the five National Societies, consisting of two members from each. A meeting of these Committees has been held, and substantial progress made toward the union of all our engineering strength in the laudable purpose of properly entertaining these guests. We are going to show them that American Engineers can get together, notwithstanding their separate organiza-

tions. We are going to try, through this Committee, to organize similar committees from representatives of the five engineering societies and from local societies in all the cities at which any number of foreigners are likely to stop. These committees are going to try and secure co-operation of the civic organizations in all these cities so that our guests may feel that they are the guests of the country and not mere unattached tourists and sightseers. It seems to me that in securing the co-operation of these societies for the first time in their history, the first great step toward a unification of the civil engineers of America has been taken.

It has not been customary on occasions of this kind to refer to sad subjects, but I feel that I will be pardoned if I turn from practical things for a moment to call attention to the loss which our Society and your Board of Direction have sustained in the deaths of Mr. Emil Gerber and Past-President Alfred Noble. Mr. Gerber was a conscientious and practical member of the Board of Direction, and cheerfully gave his time to the profession. He was a member of two Special Committees, Steel Columns and Struts, and Stresses in Railroad Track. He was also a member of the Finance Committee. We shall miss him greatly. Mr. Noble was a member of the Special Committee on Valuation of Public Utilities and was Chairman of the Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers. He was an Honorary Member of the Institution of Civil Engineers of Great Britain. For his great service in connection with the Isthmian Canal projects, this Nation owes him a debt of gratitude which can never be repaid. It should be our duty and pleasure to see that all our influence is directed toward securing, on the part of the Government, a fitting and enduring tribute to his memory.

I cannot close this address without giving expression to my deep appreciation of and thanks for the honor which was conferred upon me in selecting me for your President.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE CLARIFICATION OF SEWAGE BY FINE SCREENS.

By Kenneth Allen, M. Am. Soc. C. E.

To be Presented October 21st, 1914.

Synopsis.

The object in presenting this paper is to bring before members of the Society who are interested in sewage disposal several devices for fine screening that have been installed within the last few years, with such data regarding their merits and defects as the writer has been able to gather from various inspections and by correspondence.

A secondary object has been to indicate the relative position of fine screening as a method of sewage treatment with respect to tank treatment, in the belief that, although the field for fine screening is distinctly limited, its possibilities are not yet fully realized.

No attempt has been made to give other than general or essential details regarding the types of screens discussed.

By fine screens is meant those used in partly removing the smaller organic particles of suspended matter which cause a rapid decrease of the dissolved oxygen of the streams into which they are discharged; and, for the present purpose, such screens are assumed to have a maximum free opening of not more than $\frac{5}{8}$ in., or 15 mm.

Five types are considered. Several screens, otherwise excellent, but which are cleaned below the surface, are not considered, as this method

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

of cleaning is believed to be fundamentally bad. All the types discussed are in successful operation in Germany, and there are a limited number in England and the United States. These types are:

1.—The Band Screen, exemplified by that at Hamburg, designed by Brunotte and manufactured by the Buckau Machinery Company, of Magdeburg, Germany, and the Carshalton Screen at Sutton, manufactured by John Smith and Company, of Carshalton, England.

2.—The Wing Screen, of Uhlfelder and Schneppendahl, in use at Frankfort, Germany, and manufactured by J. Fries Sohn, of that city.

3.—The Shovel-Vane Screen, in use at Strassburg, Germany, and made by the Geiger Machine Works, of Karlsruhe.

4.—The Drum Screen, designed by G. Windschild, of Dresden-Cossebaude, in use at Trier; and that designed by O. M. Weand, of Reading, Pa., and in use at Brockton, Mass.

5.—The Riensch-Wurl Screen, used at Dresden, and manufactured by Wilhelm Wurl, of Berlin-Weissensee.

To furnish the best results, a screen should be efficient in the removal of material, reliable under conditions to be met in operation, free from nuisance, accessible for repairs, and inexpensive in first cost and operation. In general, the screens described in this paper meet these conditions quite well. Naturally, other things being equal, the finer the screen the more efficient it is, but collected data are so discordant that the desirability of standardizing the methods of obtaining and recording results is made very evident.

It is suggested that special attention be given to the mode of sampling, that samples be weighted in accordance with the flow, and that the amount of preliminary cleaning by coarse screens and grit chambers be always noted.

Except the Hamburg screen, which is somewhat coarser than the others, the fine screens now in regular operation in Germany will ordinarily remove more than 30% of the suspended solids, two-thirds or more of which consist of organic matter. This fact, which is not yet generally recognized in America, places fine screening in competition with tank treatment, although, usually, its efficiency is somewhat less.

In general, fine screening should be considered:

1. Where the removal of all but the very fine solids will furnish a satisfactory effluent;
2. Where land values are so high as to render sedimentation installations costly;
3. Where excavation for tanks would be costly;
4. Where the disposal of sewage sludge would be expensive or cause a nuisance;
5. Where the recovery of grease or fertilizing value is important.

The Removal of Suspended Solids from Sewage.

The discharge of crude sewage into streams is objectionable, primarily, on account of the grosser solids held in suspension or floating on the surface. These are offensive to the eye, and cause foul deposits on river and harbor bottoms, which produce objectionable odors and, by depleting the water of its dissolved oxygen, reduce its capacity to digest the impurities which remain.

Although these dissolved impurities as well as the pathogenic bacteria are left, by far the greater part of the nuisances from sewage effluents are due to the suspended solids. Moreover, where finishing processes are required for the oxidation of the impurities held in solution, or for sterilization, it has been found desirable to remove a part of the solids as a preliminary measure, in order to facilitate these more refined processes. Hence methods of eliminating solids from sewage are of primary importance and are being adopted quite generally by progressive cities.

The two practicable and standard methods of accomplishing this are by allowing the solids to deposit by settling in tanks, from which they are removed at intervals, and by intercepting them by screens. Ordinary sewage in American cities contains about 700 parts per million of total solids, of which 250 parts are in suspension. Of these, some 50 parts per million are so finely divided as to be either colloidal or approaching this condition, and are incapable of removal by ordinary methods. The percentage of suspended matter removable by tank treatment depends on the parts per million in the crude sewage, the length of retention in the tank, the velocity through the tank, and its

design; but, with ordinary sewage and with good design and operation of the tank, one may expect a removal:

With plain sedimentation, of about:
65% of the suspended solids, and
30% of the total organic matter.
With chemical precipitation, of about:
80% of the suspended solids, and
50% of the total organic matter.

It is commonly stated that from 15 to 20% of the suspended matter may be removed by screening, under favorable conditions. It is believed by the writer that the time has come when these figures should be revised; that modern improvements in screen design have reached the point when screening may fairly be compared with tank treatment in the percentage of suspended solids removed; in addition to which it is generally the case that an analysis will indicate a higher percentage of organic matter—or that most objectionable to discharge into streams —per pound of dried solids in screenings than in settled sludge.

Types of Screens.

In general, the screens in use consist either of parallel bars or wires, perforated plates, or woven rods or wires.

Coarse Screens.—Screens of this type are in common use before pumps to protect the valves and prevent the obstruction of water passages. In their simplest form, they consist of a series of vertical or sloping bars, usually from 1 to 4 in. apart, and are cleaned by hand (Figs. 1, 2, and 3). In some large installations the bars form the sides of a cage with a perforated bottom, which is raised by power to an operating floor where it is cleaned by hand. This form is used at the outfall works at Boston, Mass. (Fig. 4). In England, the fixed screen of sloping bars is frequently cleaned automatically by rakes which engage between the bars near the floor of the screen chamber, draw the screenings to the top of the screen, and deposit them on a band conveyor. Still another type of coarse screen is in the form of a basket of interlaced rods through which the sewage passes downward. This screen is raised for cleaning. A preferable form is a rotary screen which is cleaned above water, such as that at the Crossness Outfall Works at London (Fig. 5). In this example

FIG. 1.—COARSE SCREEN AT PROVIDENCE, R. I.

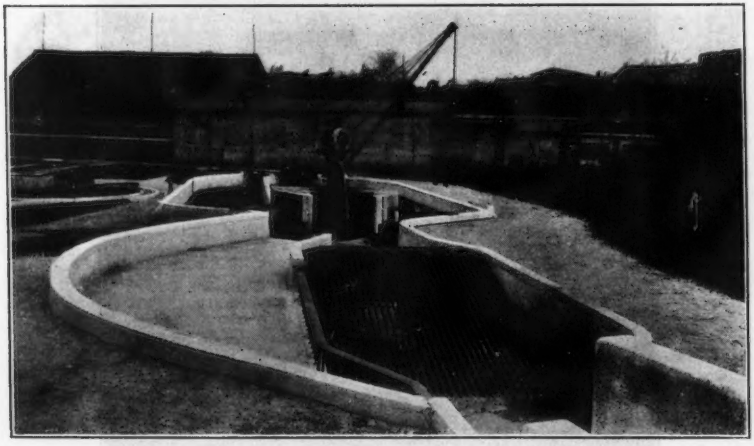
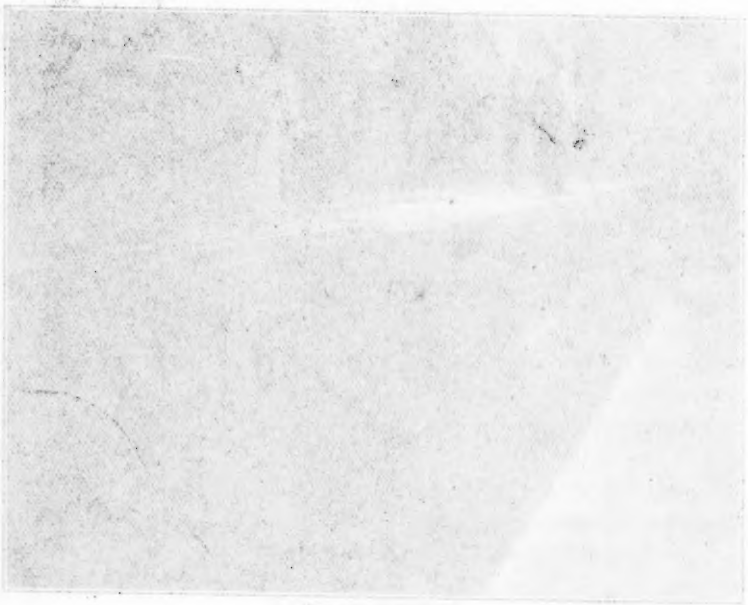
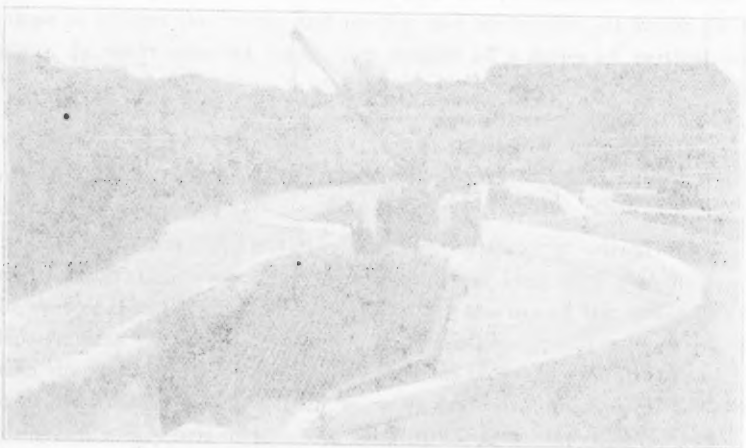


FIG. 2.—COARSE SCREEN AT ESSEN, N. W.





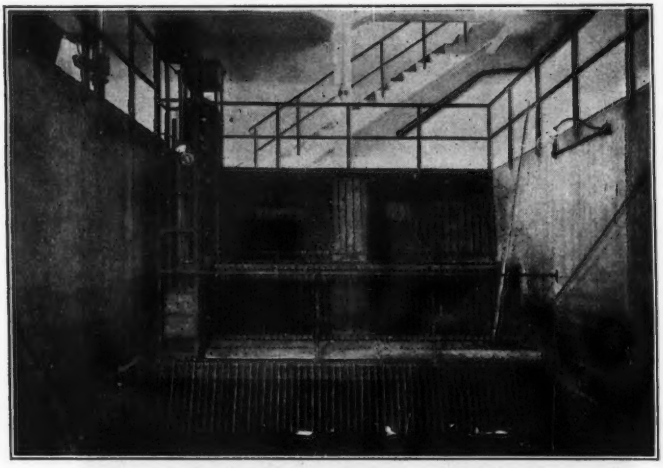
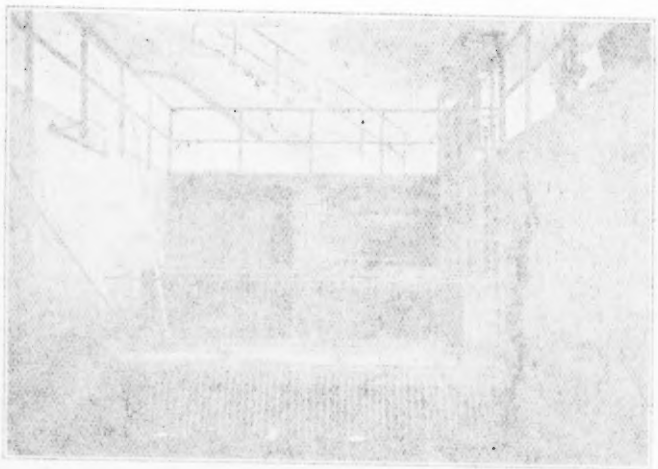


FIG. 3.—COARSE SCREENS AT COLOGNE.

FIG. 4.—CAGE SCREEN AT BOSTON, MASS.



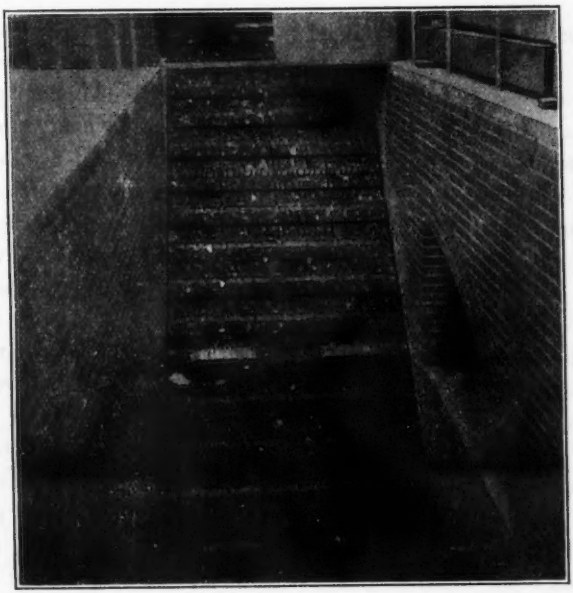
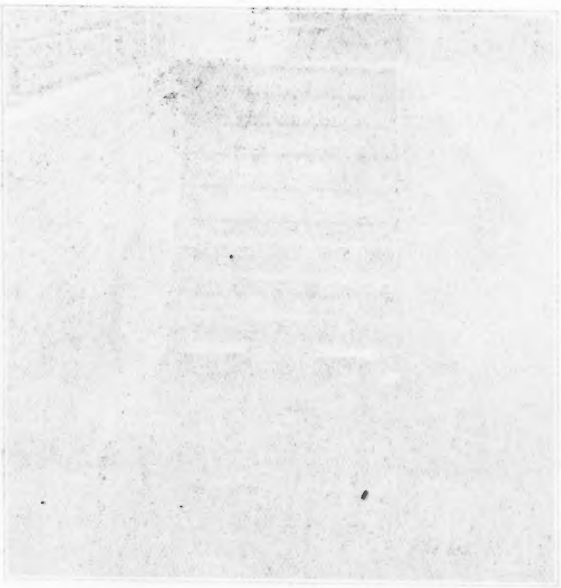
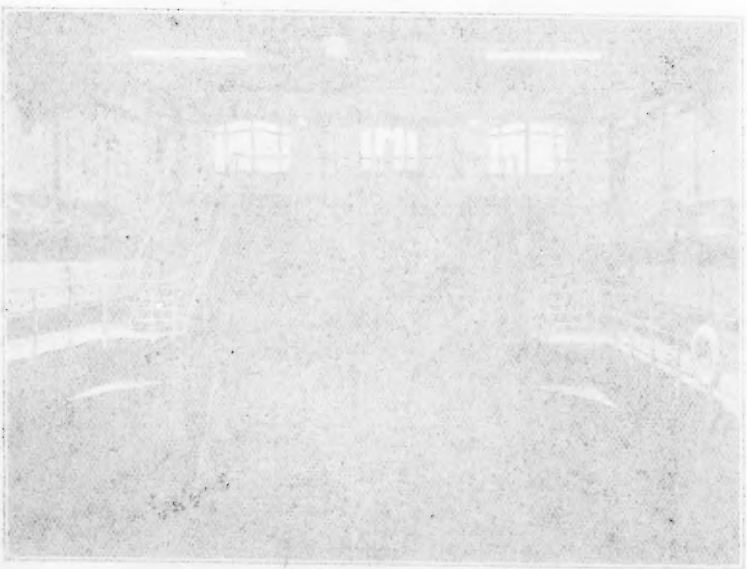


FIG. 5.—COARSE BAND SCREEN AT CROSSNESS, LONDON, ENGLAND.

FIG. 6.—BAND SCREEN AT HAMBURG.





grit also is removed by buckets attached to the screen. The spaces in this screen are ¾-in.

According to E. Kuichling, M. Am. Soc. C. E., coarse screens generally remove from 3 to 10 cu. ft. of material per million gallons of sewage, equivalent to about 104 lb. of dried matter or 12½ parts per million of the suspended solids.

Fine Screens.—These screens are in use to a limited extent, both in America and England, but their most marked development has been brought about in Germany. This is perhaps chiefly due to the fact that, though there are many large cities in that country, they are mostly on large streams, such as the Elbe and Rhine, so that, in many cases since 1903, the Government has sanctioned processes which merely retain solids more than 3 mm. in diameter. In such cases, screens may be substituted for the more expensive tank installations. The result has been an introduction of several types of fine screens in many towns during the past few years.

It is not easy to draw a definite line between fine and coarse screens, but, for the present discussion, we may consider those having a clear opening or mesh not greater than 0.6 in., or 15 mm. as "fine".

From experiments by Monti at Berlin, it was concluded*

"that most of the suspended matter in sewage is usually in a very fine state of division, or capable of passing through a sieve having meshes 0.02 inch square; that most of the solid fecal matter found in the liquid can be arrested on a sieve having meshes 0.25 inch square; and that a sieve with meshes 0.10 inch square is probably fine enough to extract from the sewage all suspended matter of appreciable magnitude, and especially such as is likely to become offensive [under conditions proposed for Rochester]. * * * It is believed, * * *, that ample evidence has been submitted to show that a surprisingly large quantity of offensive organic matter can be extracted from sewage by properly designed screening appliances, and that the screened liquid may then be admitted safely for final disposal into bodies of water which would soon become repulsive without such screening."

Function of Fine Screens.

The function of fine screens is quite different from that of coarse ones, so that we may be justified in considering them in a class by themselves, presenting problems of design and operation that have no similarity to the design or operation of coarse screens. The

* Sewage Disposal System. Rochester, Kuichling, 1913.

use of fine screens may be advisable for any of the following purposes:

1. The removal of matter that otherwise would cause the pollution of the stream receiving the effluent;
2. The removal of solids that otherwise would cause the frequent subsequent clogging of sprinkler nozzles or filters; and
3. The removal of material likely to promote the formation of scum in subsequent tank treatment.

In Germany their use has been justified mainly for the first reason, but it is recognized that fine screening before applying sewage to sprinkling or sand filters may cause a marked reduction in operating expenses. In Baltimore fine screens have been placed between the tanks and sprinkling filters, with the result that whereas, previously, "two men, eight hours per day, were always required and sometimes more for cleaning nozzles for between six and nine acres of filter area, with the screen in operation two men are required to spend about two hours a day each for the same number of nozzles."*

According to the Massachusetts State Board of Health: "When practically all the suspended matters were removed from the sewage the possible rate of filtration [to sprinklers] could be doubled."†

On the other hand, their use for this purpose has been discontinued at Birmingham, England, and at Atlanta, Ga., as unnecessary after efficient tank treatment. On the whole, it is questionable whether their expense is often justifiable for this purpose. Mr. Francis E. Daniels‡ states that "the use of fine screens will entirely prevent the formation of tank scum." If this is substantiated by further experience, it will be an argument in their favor in connection with those sedimentation plants where the formation of scum has been found to be a nuisance.

Conditions to be Satisfied.—The conditions to be satisfied in a fine screen may be summarized as follows:

a. It must remove a sufficient quantity of the suspended solids and total organic matter in regular operation to satisfy the

* Letter from H. C. McRae, Assoc. M. Am. Soc. C. E., Division Engineer, Baltimore Sewerage Commission, February 24th, 1914.

† *Surveyor and Municipal and County Engineer*, March 6th, 1914.

‡ Director, Water and Sewerage Inspection, State Board of Health, New Jersey, *Municipal Journal*, January 15th. 1914.

needs of the situation. In other words, its efficiency must be adequate.

b. In operation, it must be reliable; and, in capacity, it must be ample, under the conditions of sewage flow and surface elevation which may obtain.

c. Its operation must be free from nuisance and unsanitary conditions.

d. All parts must be accessible for inspection, repair, and renewal.

e. The cost of installation must be low.

f. The costs of operation, renewals, and repairs must be low.

These conditions will be considered subsequently, in their relation to different screens, under the heading, "Comparison of Screens."

Types of Fine Screens.

The fine screens in successful operation may be classified under five distinct types, all of which are found in Germany. In all of them the screenings are raised gently above the surface before being removed, instead of being raked off while submerged, as is done with some of the coarser screens in England, and at Paris, Cologne, and elsewhere.

These types are as follows, in the order of the usual size of their opening (Fig. 7)*:

1. The band screen (*Abfischgitter*), consisting of an endless flexible band, either of wire mesh or links, which passes over upper and lower rollers.

2. The wing screen (*Klärrechen*), formed of vanes, as in a paddle-wheel, which are composed of radial bars at uniform distances apart.

3. The shovel-vane screen (*Siebschaufelrad*), similar to the wing screen, but with semicircular wings and a different method of removing the screenings.

4. The Riensch-Wurl screen (*Separatorscheib*), which consists of a perforated disk surmounted by a truncated cone, also perforated. The disk is mounted on an inclined shaft.

5. The drum screen (*Siebtrommel*), consisting of a cylinder or cone of perforated plates or wire mesh, which rotates on a horizontal axis.

* Except that the wire mesh used in any type may be very fine.

The fourth and fifth may have openings of 1 mm. or less. Descriptions of each, with typical example, are given.

Band Screens.

Band screens may be broadly divided into two classes: (*a*) those composed of bars or links; and (*b*) those composed of a fine mesh.

a.—Hamburg.—The Hamburg screen (Fig. 6) at the main outfall works on the north side of the Elbe, is composed of a series of parallel links, 14.2 in. (36 cm.) long, 0.2 in. (0.5 cm.) thick, and

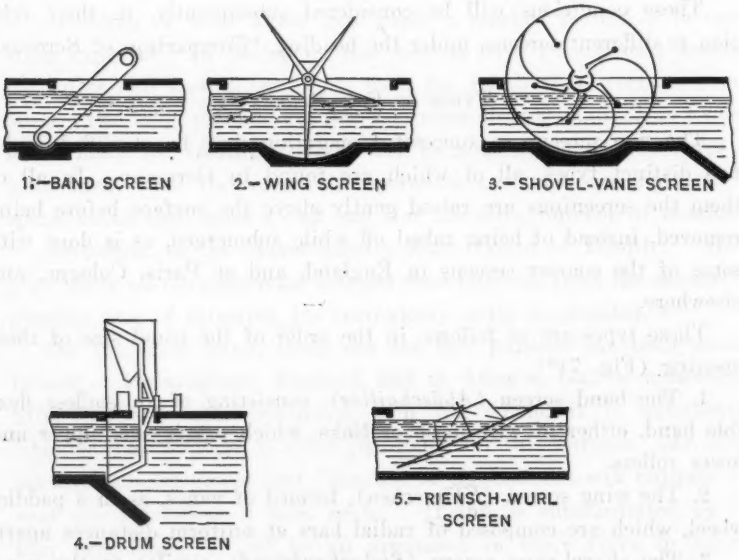


FIG. 7.

0.59 in. (15 mm.) apart. These links were first made of hard rubber, but are now being made of aluminum alloy, the essentials being a material of sufficient strength, not readily corroded, and not too heavy. There are some 14 000 links in the screen. They are assembled in sections which are held together by angle-iron frames, 9.85 ft. (3 m.) long and 15 in. (38 cm.) wide, of which there are forty-six in each of the two screens installed. The bars are readily removable from the frames for purposes of repair.

Each of the two screens at this station is about 33 ft. (10 m.) long, 10.8 ft. (3.3 m.) wide, and 19 ft. (5.8 m.) high, over all. Their inclination to the horizontal is about 63 degrees. This great length has been made necessary by the tidal range of 6.2 ft. to which the screens are subjected. They are supported and operated by wheels resting on rails, on each side, which pass over sprocket wheels mounted on a shaft at each end, the upper shaft being connected by gearing to an electric motor.

As the assembled frames of links forming the screen band move upward with a velocity of from 1 to 2 in. per sec., they carry with them, above the surface of the sewage, the material intercepted. As larger particles of material are caught, they serve to prevent the passage of much that is less in size than the clear space between the links. At times paper becomes plastered over the links, and backs up the sewage as much as 28 or 30 in. (70 to 75 cm.), instead of some 8 in. (20 cm.), the usual head. For this reason the screen is designed to resist this occasional horizontal thrust. The ordinary depth on the screen is about 6 ft.

Back of the screen, near the top, hangs a frame to which are attached at right angles the prongs of a comb scraper or rake; these enter the spaces between the links intermittently and remove the screenings. By a contact with an electro-magnet, this frame swings back after cleaning a section of the screen, and the screenings are scraped off automatically to a band conveyor by the edge of a plain, hard-rubber scraper. The frame then swings forward again and repeats the operation with the next section of screen.

For convenience in making repairs, the entire screen can be swung up above the sewage by a chain attached to its lower edge.

About 2½ h.p. is required in the ordinary operation of the screen, and 2½ h.p. more for the cleaning device. The station contains two 30-h.p. generators, which furnish current for the following motors:

Bucket elevator in grit chamber	4.	h.p.
Swinging the bucket elevator in grit chamber	4.6	"
Screen and belt conveyor	3.	"
Inclined screw conveyor	4.5	"
Bucket elevator for screenings	1.75	"
Ventilator	3.	"

In addition, current is furnished for 4 arc lamps, 74 incandescent lamps, and for feeding a storage battery used for the illumination of boats in the main outfall sewer.*

The population now tributary to the works is about 1 000 000 and the sewage flow is 53 000 000 gal.† (190 000 cu. m.) per day. This varies from 140 cu. ft. per sec. (4 000 liters per sec.) during dry weather to 700 cu. ft. per sec. (20 000 liters per sec.) during storms.

On entering the station the sewage first flows through a grit chamber, 54 ft. long, 30 ft. wide, and about 6½ ft. deep below the normal surface of the sewage. About 9 cu. yd. (7 cu. m.) of sand and other heavy material are removed per day here, or 0.17 cu. yd. per million gallons.

From the grit chamber the sewage passes through the screens. These take out 18 cu. yd. (14 cu. m.) per day or 0.34 cu. yd. per million gallons of sewage. It then flows through tide-gates to three riveted-steel outlet pipes, 6.56 ft. (2 m.) in diameter, and 230 ft. (70 m.), 328 ft. (100 m.), and 436 ft. (133 m.) long, respectively, from which it is discharged near the bottom of the Elbe.

Operation is suspended for about 2½ hours during each high tide, or 5 hours per day, as the surface of the river is then higher than that of the sewage, causing the tide-gates to close.

The grit and screenings are carried by scows, having a capacity of 90 cu. yd. (70 cu. m.), 19 miles down the Elbe to the Island of Waltershof, where they are removed by farmers and used to fertilize land, on payment of 7 cents per cu. yd. by the city.

TABLE 1.—Composition of Grit and Screenings from Hamburg Screens.

Analysis No.	Grit.			Screenings.		
	1	2	3	1	2	3
	%	%	%	%	%	%
Water	65.35	55.90	22.87	83	92.62	85.24
Nitrogen	0.52	0.27	0.21	0.78	0.27	0.02
Phosphoric acid	0.52	0.29	0.40	0.42	0.18	0.12
Fats	1.83	1.28	0.44	7.88	1.68	5.11
Ash	20.28	37.15	71.28	1.33	1.10	5.16
Of which, Sand	.....	35.33	67.86	.....	0.32	2.88

* "Die Kanalisationsanlagen der Freien und Hansestadt Hamburg," Curt Merckel, Hamburg, 1910.

† 190 000 cu. m. from 950 000 population in 1910, "Die Stadtische Abwasserbeseitigung in Deutschland," Salomon, 1910, Vol. III, p. 230.

Table 1 shows the composition of the grit and screenings, as given by Merckel.

From analyses of five samples of mixed detritus of varying character from the grit chamber and screens, Dr. Gillmeister, City Chemist, has estimated the value of the contained nitrogen and phosphoric acid as fertilizer, as given in Table 2.

TABLE 2.—VALUE OF CONTAINED NITROGEN AND PHOSPHORIC ACID AS FERTILIZER.

Sample No.	Percentage of moisture.	Percentage of N in dried material.	Percentage of P_2O_5 in dried material.	Percentage of grease in dried material.	Value of Dried Material.		Description of material.
					In dollars per ton.	In Marks per 100 kg.	
I	92.62	3.69	2.34	22.85	$16.87	7.81	Loose; lumpy; somewhat plastic, sticky; fecal odor.
II	22.87	0.27	0.52	0.57	1.62	0.75	Blackish; sandy; mixed with particles of wood and coal.
III	55.90	0.61	0.67	2.91	3.09	1.43	Blackish; sandy; fine; odor slightly fecal.
IV	85.24	0.10	0.72	34.63	1.16	0.54	Loose and fibrous; slightly sandy; slight odor.
V	59.79	1.06	0.83	1.98	5.03	2.33	Rather loose, sandy; dusty.

When mixed with four parts of street sweepings, the grit and screenings are disposed of by incineration.

The first cost of this plant is given by Merckel as follows:

Foundations, etc.	$174 000
Screen house	10 500
Machinery	33 300
Storage house at dock	2 900
Outlet pipes	53 500
Total	$274 200

Up to the present time, it has cost about $300 000.

These screens were designed by Brunotte, and are manufactured by the Buckau Machine Fabrik, of Magdeburg-Buckau.

Four similar screens, although only about 6 ft. (1.8 m.) wide and with 0.4-in. (10-mm.) spaces between the links, have been installed more recently on the south side of the Elbe. These are

operated by Diesel motors, and require only about 1 h.p. each. The present population tributary to this plant is about 10 000, increased during the day by 20 000 workmen who live on the north side of the river.

The grit chamber and screens together remove about 11.7 cu. yd. (9 cu. m.) of material daily, one-third of which is organic, indicating a higher efficiency than in the older works. This is probably due in part to a lower velocity (4 in. = 10 cm. per sec.) in the grit chamber than at the North station (19.7 in. = 50 cm. per sec.).

The South Hamburg station is designed to care for three times the present flow.

b.—Göttingen.—Other band screens, in which a wire mesh takes the place of links, are used in a number of places. There are two such at Göttingen, installed in 1903, which will serve as an illustration.

Here the bands are 55¾ ft. (17 m.) long, and are set at an angle of 55° with the horizon. They are composed of 0.06-in. (1½-mm.) copper wire, twisted spirally, and woven with a 0.4-in. (10-mm.) mesh, which passes over two drums, 20 in. (50 cm.) in diameter. The upper drum is belt-driven from an undershot wheel, and the lower one is placed in a sump, 28 in. (70 cm.) deep. Brass angles are attached to the screen, 3.3 ft. (1 m.) apart, to catch matter tending to roll back into the sewage. The screen is cleaned by a brush and water jets.

The material is delivered to a tip-wagon having a double interior screen on one side for draining off the moisture, and is then composted with peat dust and sweepings for a fertilizer.

In operation, the screen moves with a velocity of about 8 ft. per min. (4 cm. per sec.)

The lower end of the screen can be raised above the surface, as at Hamburg, for inspection and cleaning. It is also lifted at night when a 2-in. (5-cm.) bar screen above is lowered to take its place.

As the band stretches, it has been found necessary to tighten it from time to time. This results in a slight narrowing, so that wooden fillers are inserted to prevent leakage between the screen and the sides of the channel.

Every 5 or 6 weeks the screens require a thorough cleaning, and the sediment that has collected in the channels is flushed out.

As Göttingen is sewered by the separate system, grit chambers have not been found necessary. In 1907, when the population was 35 000, 38% of the houses were supplied with water-closets. The screenings amounted to 10.3 cu. yd. (7.9 cu. m.) per 100 population.* The screenings per 1 000 population per day were, therefore, $\frac{103}{365} = 0.282$ cu. yd. The sewage flow was then about 2 600 000 gal. per day,† so that about 3.8 cu. yd. were removed per million gallons. The suspended matter is said to have been reduced 90 per cent. The total cost of the works was $18 500 (60 700 Marks), of which the screens and machinery cost $3 600 (15 000 Marks). The screens were installed by City Engineer Hertzberg.

There are two other fine band screens to be mentioned.

The Carshalton Screen.—This screen (Fig. 8) is similar in principle to those at Göttingen, but the slope is not so great—usually 30° with the horizon—and it is "made of perforated steel strips, twisted steel wires and pins, similar to a coal picking belt. The holes are usually $\frac{3}{8}$ in. in diameter." It is operated by an undershot wheel placed behind the screen.

These screens are installed at Birmingham, Chester, Croydon, Wrexham, Ipswich, and about a dozen more English towns, generally between tanks and oxidizing beds. They are manufactured by John Smith and Company, of Carshalton. According to the manufacturers, they will remove about $2\frac{1}{2}$ cu. yd. of material per million U. S. gallons of sewage, varying, of course, with the character of the latter. A screen with a 6-ft. width will care for about 2 750 000 U. S. gal. in 24 hours. At Sutton, one of these screens, with a $\frac{1}{4}$-in. mesh, removes 1 200 lb. of screenings per million gallons.‡

The Jennings Screen.—This screen (Fig. 9) consists of a fine wire-mesh band, and has been in operation since July, 1913, at the Chicago Stock Yards, with a flow of from 1 250 000 to 1 500 000 gal. per day. Although this may be said to be still in the experimental stage, a brief reference to it will not be out of place. The wires are of Monel metal, with 40 meshes per inch. The screen is composed of separate sections of removable frames, for convenience in

* Based on the weight of screenings, assuming their specific gravity to be 1.1.

† 2 200 000 gal. per day from 30 000 persons in 1905, E. Kuichling, *Proceedings*, Fourth Annual Meeting, New Jersey Sanitary Association, 1908.

‡ Sewage Disposal System, Rochester, N. Y., E. Kuichling, April, 1913.

repairs and renewals. The screenings are blown off by air compressed to 1½ lb. per sq. in. at the slotted jet used for cleaning. A 10-h.p. motor is used for operating the screen and blower, although less power is required after starting. The entire cost for air and power amounts to from 12 to 15 cents per hour. The plant is looked over once or twice a day by one of the regular employes so that there is no extra charge for attendance.

According to C. A. Jennings, Assoc. Am. Soc. C. E., the designer, "with an influent running about 700 p.p.m. suspended matter, the screen removed about 63% * * *. The screenings contained an average of about 79% moisture."

In one test of 6 hours' duration, the wet screenings amounted to 6 186 lb. per million gallons. Another test of 5 hours gave 4 828 lb. per million gallons, the suspended matter in this case being 20% lower than in the former.

Features of this screen are the fineness of the mesh and its construction in sections, as at Hamburg.

Wing Screens.

Frankfort.—Three screens of five vanes each are installed at Frankfort (Fig. 10). The vanes are 6.56 ft. (2 m.) wide and 9.84 ft. (3 m.) long, measuring from the center of the shaft. The effective radial length is 8.2 ft. (2.5 m.). Each vane consists of 0.16 by 1.20-in. (4 by 30-mm.) steel bars, 0.4 in. (10 mm.) apart. These bars are assembled in two sections, 3.28 ft. (1 m.) wide, and are tied together by nine transverse rods. On these rods filler rings alternate with the bars to preserve the proper spacing, and, by changing the size of these rings, the spacing may be altered. The ratio of the free opening to the entire vane is 1 : 2.4. To avoid undue friction in the passage of the sewage through the screen, these bars are given a less depth between the holes for the tie-rods.

The rectangular channel is depressed at the screen, so that during a revolution one vane will occupy the entire cross-section of the stream at all times and prevent the by-passing of an appreciable quantity of sewage.

As the lowest vane, moving against the current, rises slowly, a hard-rubber scraper, hung by a long arm at each end, is forced by the motion of the vane to travel from its inner portion near the

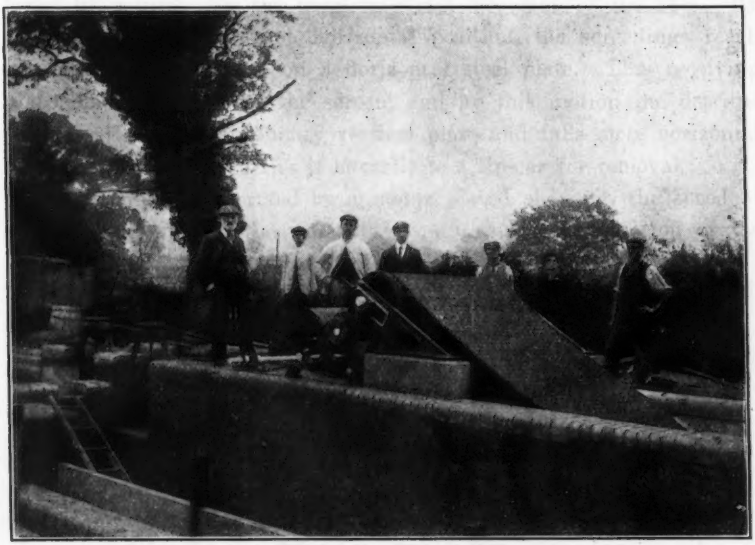
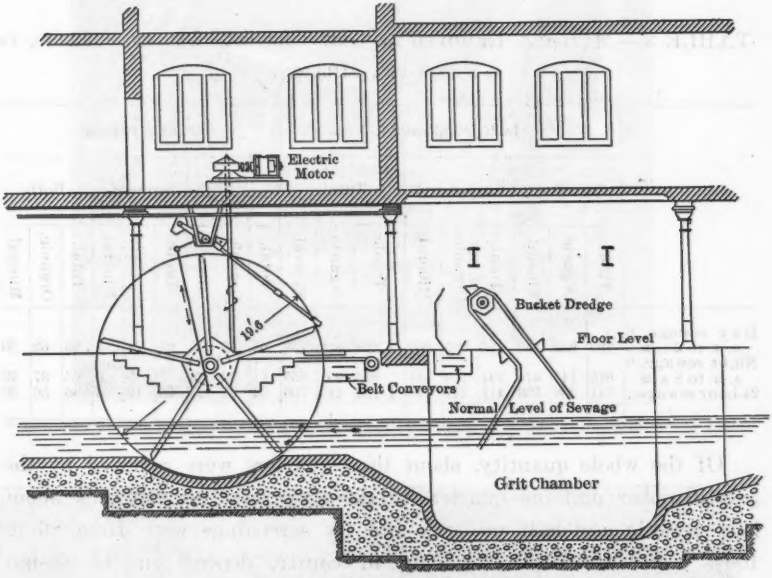


FIG. 8.—CARSHALTON SCREEN, AT WREXHAM, ENGLAND.

FIG. 9.—THE JENNINGS SCREEN.

shaft to the extremity of the bars. This scraper is, or may be, followed by a brush for the more thorough cleansing of the bars. By the time the vane is in a horizontal position, the screenings reach its outer edge and drop on a horizontal steel plate. This receiving plate then recedes from the screen, and by this motion the detritus is scraped off by a stationary vertical plate and falls on a horizontal belt conveyor. This carries it laterally to a tip-car for removal.

Each screen is operated by a motor placed above it, the speed of which is adjusted to the work to be performed. The motion of the



WING SCREEN AT FRANKFORT

FIG. 10.

receiving plate is transmitted by mechanism from the scraping apparatus, so that it occurs at precisely the time required. The belt conveyor has a velocity of 3¼ ft. (1 m.) per sec.

The sewerage of Frankfort is on the combined system, and the population is about 420 000. The volume varies from 21 000 000 to 26 000 000 gal. (80 000 to 100 000 cu. m.) per day. The daily per capita flow, therefore, is about 56 gal. (213 liters).

After passing a grit chamber, where 20 tons of detritus are removed daily, or, say, 0.8 ton per million gallons, the sewage flows

through the screens, two of which are generally in use. They remove automatically 50 tons of screenings per day, or 2 tons per million gallons of sewage, composed largely of feces and paper.

Measurements made about 7 years ago indicate that 16% of the suspended matter is removed in the grit chamber and 10% more by the screens, making 26% in all. At that time the water supply was 47 gal. and the sewage flow 39½ gal. per capita. Table 3 shows the quantities of material removed by the grit chamber and the screens, as given by Uhlfelder and Tillmans.*

TABLE 3.—MATERIAL REMOVED BY GRIT CHAMBER AND BY SCREENS, IN PARTS PER MILLION.

	In Raw Sewage:									Intercepted by:								
	Dissolved.			Suspended.			Total.			Grit Chamber.			Screens			Both.		
	Total.	Organic.	Mineral.	Total.	Organic.	Mineral.	Total.	Organic.	Mineral.	Total.	Organic.	Mineral.	Total.	Organic.	Mineral.	Total.	Organic.	Mineral.
Day sewage, 8 A. M. to 2 A. M..	783	230	553	485	285	200	1 268	515	753	65	39	26	28	24	4	93	63	30
Night sewage. 2 A. M. to 8 A. M..	615	141	474	191	110	81	806	251	555	61	32	29	5	5	0	66	37	29
24-hour sewage..	741	208	533	411	241	170	1 152	449	703	64	37	27	22	19	3	86	56	30

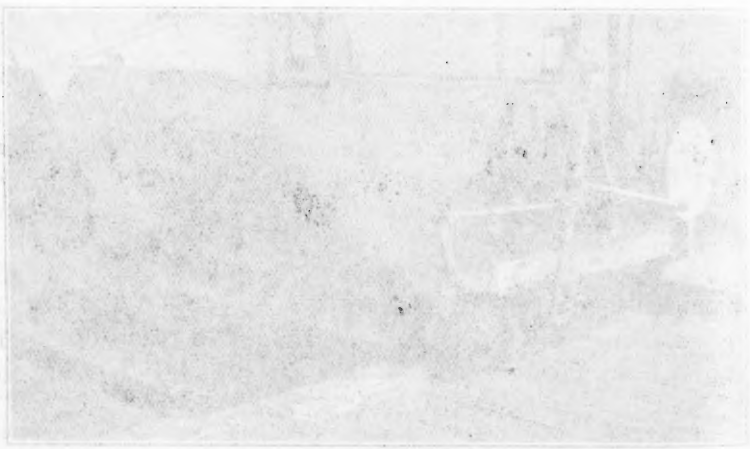
Of the whole quantity, about three-quarters were removed by the grit chamber and one-quarter by the screens. The grit was about half organic and half mineral, and the screenings were from 86 to 100% organic. These relations, of course, depend on the design and operation of both grit chamber and screens; but, at Frankfort, although the grit chamber removes three times as much material as the screens, the latter remove half the organic matter taken out. To operate one of these screens requires 2.3 h.p.

During eight years of service, the repairs and renewals are said to have been insignificant. The annual cost of operation is given by the late City Engineer Koelle as 1.1 cents per capita. It is otherwise stated as 18 cents per cu. yd. (0.75 Mark per cu. m.) of screenings, or about one-sixth of the cost when removed by hand.

*"Die Frankfurter Klaranlage," *Wasser und Abwasser*, Vol. 1, No. 7, 1909. *Mitteilungen a. d. Königl. Pruf. für Wasserversorgung und Abwasserbeseitigung*, Vol. 10, 1908.

FIG. 11.—WING SCREEN AT ELBERFELD.

FIG. 12.—WING SCREEN AT BRADFORD, ENGLAND.



Elberfeld.—At Elberfeld there are two screens (Fig. 11), similar to those at Frankfort, taking the sewage from a population of about 340 000 persons in Elberfeld and Barmen. The sewers are mostly on the separate system, and the volume of sewage is 16 000 000 gal. (60 000 cu. m.) daily. It contains 500 parts per million of suspended material.

The sewage is quite fresh when received at the works. It first enters a grit chamber where two dredges take out the coarse material—largely kitchen refuse, with a little sand. It then passes the screens. These rotate once in 3½ min. and, with the grit chamber, remove 18.2 cu. yd. (14 cu. m.) of material per day, or 1.15 cu. yd. per million gallons. As at Frankfort, these appeared to consist chiefly of feces and paper. They are placed in unprotected dumps for drying, and are naturally very offensive in warm weather.

The screens are cleaned about once a week, while in operation, by raking out the bars by hand. This takes one man about ½ hour for the two screens.

The power required for operating a grit elevator, a sludge pump, and the screens is usually 5 h.p., but, for short periods, may be as much as 10 h.p.

The Elberfeld screens are placed before settling tanks, the excuse for this preparatory treatment being the removal of a substantial part of the solids in a form containing relatively little moisture and, therefore, readily handled.

At the works of J. S. Fries Sohn, at Frankfort, where the Frankfort and Elberfeld screens were manufactured, the writer was told that one such screen would ordinarily remove 16 cu. yd. (12 cu. m.) of material from 8 000 000 gal. of sewage (30 000 cu. m.) per day, or 2.0 cu. yd. per million gallons, and that for operating the screens 3 h.p., or, with auxiliary machinery, 5 h.p., would be required.

Bradford, England.—Two screens of the Frankfort type were installed at Bradford a few years ago, but differ from the former in some details (Fig. 12). Each is 6 ft. 2½ in. wide, with length of vane of 4 ft. 11½ in. and a capacity of 3 500 000 gal. per day. The bars are about ¼ by 1 in. in section, and ½ in. apart. There are no cross-rods except near the extremity of the vane. A chief function of these screens is to intercept wool fiber, which occurs in considerable quantity in the Bradford sewage. For this purpose the wing

screen has proved well adapted. In operation, the screen comes to rest when a pair of vanes reaches the horizontal position, and a rake with teeth interlocking with the screen bars is introduced and withdrawn mechanically, removing the material intercepted. A few fibers clinging to the cross-rod occasionally require removal by hand. For each one-sixth of a revolution 33 sec. are required, and an equal interval for cleaning.

Wiesbaden.—At Wiesbaden a screen of this type, with six vanes, was put in by Schneppendahl in 1899. The vanes are 6.56 ft. (2 m.) wide, and the bars are 0.6 in. (15 cm.) apart. It is operated by hand, day and night. The sewage amounts to about 3 000 000 gal. (11 000 cu. m.) daily from 100 000 persons, is domestic in character, and contains much soap and fats. It first passes a fixed screen with 1.6-in. (40-mm.) spaces. The fine screen removes 3.4 cu. yd. (2.6 cu. m.) of screenings, largely feces, or 1.1 cu. yd. per million gallons.

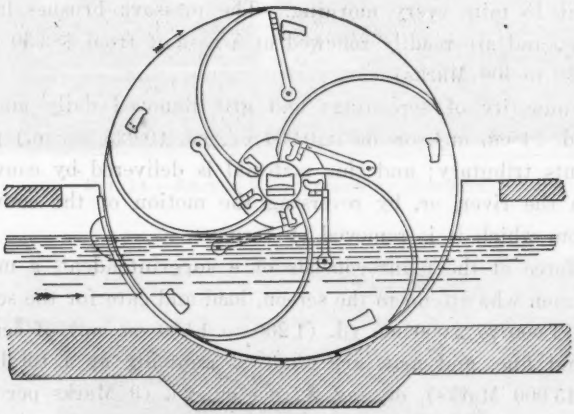
The Shovel-Vane Screen.

A very ingenious modification of the wing screen is that known as the *Siebschaufelrad,* patented and manufactured by the Geiger Manufacturing Company, of Karlsruhe (Fig. 13). In this screen there are five semicircular vanes between two circular plates which form the sides of the apparatus. The axle is hollow, with the top third open, and contains a belt conveyor. Owing to the shape of the vanes, which are nearly tangent to the circumference of the side plates on their outer extremities and to the radius at their inner edges, the sediment and floating matter are scooped up gently above the surface of the sewage, with little loss or disintegration, and finally drop by gravity from the screen to the belt conveyor. Such material as adheres to the vanes is brushed off mechanically when above the axle by a brush on the end of an arm swinging from a center near the middle of the adjacent vane. This brush is of piassava fiber, which is well adapted to the purpose. By a mechanical device it comes into contact with the vane only while swinging in an inward direction over the part of the vane above the axle. At other times, it rests near the axle, and, therefore, is ordinarily above water. In the operation there is little tendency to crush the screenings or press them through the screen itself.

The screen vanes are of pressed wires of V-section, and are 1, 2, or 3 mm. apart, and held in place by a central rib and by round transverse tie-rods.

Power is transmitted to the wheel by gearing. The motion is slow and uniform. The moving parts are open to inspection and repair above the level of the sewage.

Paper, which might partly clog the screen and obstruct the flow, is retained by a baffle placed before the screen, but sand or gravel which may pass the grit chamber above is handled and removed with the other screenings. There is a clearance of only 0.04 in. (1 mm.)



GEIGERS SHOVEL-VANE SCREEN

FIG. 13.

between the screen and the curved bottom of the channel, and this space is prevented from clogging by the constant motion of the screen against the current.

Shovel-vane screens are in operation at Strassburg and Gleiwitz, Germany, and at Temesvar, Hungary.

Strassburg.—A population of 160 000 contributes daily 11 900 000 gal. (45 000 cu. m.) of rather weak domestic sewage containing trade wastes from the city and the suburbs, Neudorf, Kronenburg, and Königshofen-Gruneberg. It arrives while quite fresh, and first passes two vertical screens of bars 0.32 in. (8 mm.) apart, which are cleaned by a movable scraper.

Below these bar screens a Geiger screen was put in as an experiment in August, 1908, with 0.10-in. (2½-mm.) openings and a width of 5.25 ft. (1.6 m.). Having been erected in an old grit chamber, the ratio of submergence to depth of sewage is said to have been unfavorable for the best results, the diameter too small, the axle too low with reference to the surface, and the free opening too small, considering the width of the channel above, which is 9.2 ft. (2.8 m.).

Nevertheless, the capacity of the screen, with the sewage backed up 12 in. (0.3 m.), is about 18 000 000 gal. per day (800 liters per sec.). The small quantity of hair and paper which collects on the screen is removed by a jet from a hose operated by a plunger pump for about 15 min. every morning. The piassava brushes last about 2 months, and are readily renewed at a cost of from $83.50 to $95.00 (from 350 to 400 Marks).

The quantity of screenings and grit removed daily amounts to 5¼ cu. yd. (4 cu. m.), or about 0.034 cu. yd. (0.025 cu. m.) per 1 000 inhabitants tributary; and this material is delivered by conveyors to scows in the river, or, by reversing the motion of the conveyor, to a pit from which it is removed by farmers.

The force at the plant consists of a superintendent, a machinist, and two men who attend to the screen, load and care for the scows, etc.

From 1 600 to 2 000 cu. yd. (1 200 to 1 500 cu. m.) of screenings, coarse and fine, and grit, are removed annually at a total cost of $3 570 (15 000 Marks), or, say, $2 per cu. yd. (9 Marks per cu. m.).

The cost of operation of the fine screens, exclusive of labor, is 30 cents (1.25 Marks) per 10-hour day, and the power consumed is 25 kw.

About 1.3 cu. yd. (1 cu. m.) of fine screenings are removed per day from the sewage of 30 000 persons, at a cost of 5.4 cents per cu. yd. (0.30 Mark per cu. m.).

Gleiwitz.—The population of Gleiwitz is about 67 000, and produces a volume of sewage varying from 1 000 000 to 2 600 000 gal. (4 000 to 10 000 cu. m.) per day. Two Geiger screens (Fig. 14), 12.5 ft. (3.8 m.) in diameter and 5.9 ft. (1.8 m.) wide, are installed. The screen vanes are composed of V-shaped rods, 0.12 in. (3 mm.) apart. The quantity of material removed per year is 4 700 cu. yd. (3 600 cu. m.), or 0.19 cu. yd. per 1 000 persons daily, at an annual cost of from $480 to $710 (2 000 to 3 000 Marks). The cost, therefore, is

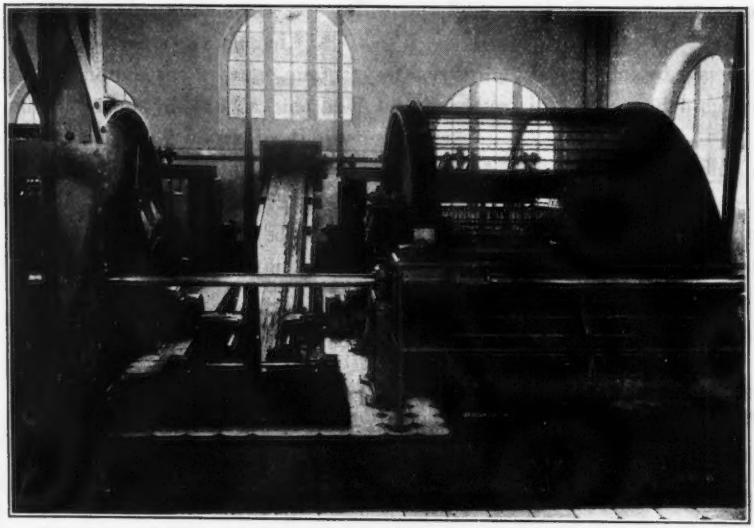


FIG. 14.—SHOVEL-VANE SCREEN AT GLEIWITZ.

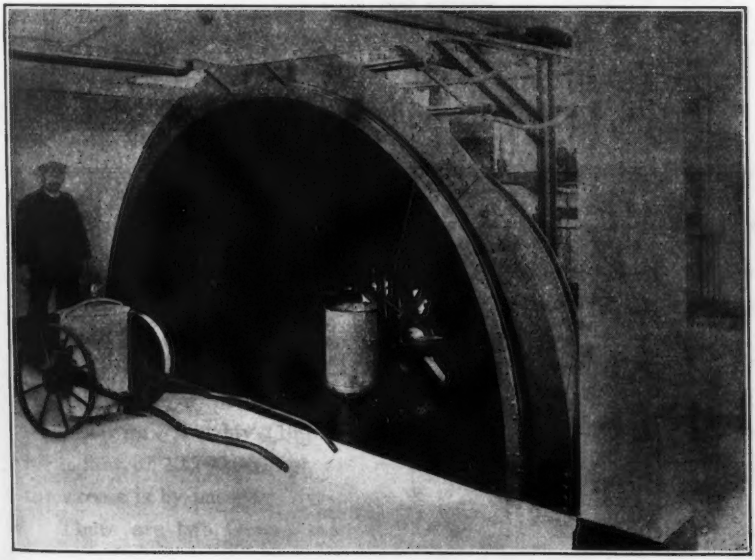
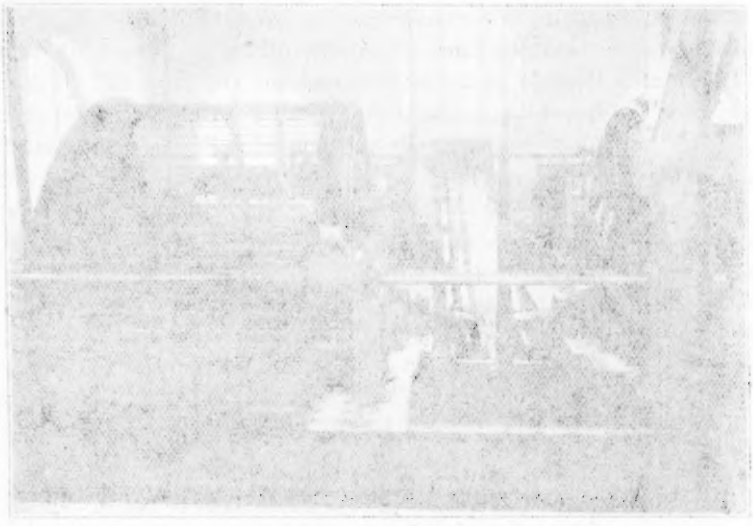


FIG. 15.—DRUM SCREEN AT TRIER. FRONT VIEW.



probably about 90 cents per million gallons, and the cost of screenings, 12½ cents per cu. yd.*

Although the average quantity of screenings is 12.9 cu. yd. per day, about 21 cu. yd. (16 cu. m.) have been removed in 1 day, and 5 cu. yd. (4 cu. m.) in 2 hours, by the two screens.

The cost of each screen was $2 800 (11 750 Marks), and that of the entire plant $6 860 (28 800 Marks).

Two men are required to attend the screens and pumps, one during the day and one at night. The power for operation varies from 1 to 2 h.p.

Analyses of the sewage before and after screening are given in Table 4.

TABLE 4.—ANALYSES OF SEWAGE AT THE GLEIWITZ SCREENING PLANT, IN PARTS PER MILLION.

	Before screening.	After screening.
Reaction	Weakly alkaline	Weakly alkaline.
Appearance	Brown, strong, with coarse material; turbid, with light settleable particles.	Yellowish brown, turbid, with fine suspended matter.
Suspended solids	2 112.8	785.2
Dried residue after filtration	1 237.2	1 260.4
Loss on evaporation	400.0 = 33.08%	388.8 = 30.85%
Oxygen consumed	105.5	100.6
Ammonia	30.0	30.0
Chlorine	262.2	279.4
Sulphuric acid	Present.	Trace.
Nitrous acid	None.	None.
Nitric acid	None.	None.

The clarifying efficiency, therefore, was $\frac{2\,112.8 - 785.2}{2\,112.8} = 62.9\%$; on the other hand, the solids in solution (probably colloidal matter) increased from 1 237.2 to 1 260.4 parts per million, or about 1.9 per cent.

Temesvar.—The tributary population at this place is about 60 000, and the rate of flow through the screens varies from 3 650 000 to 9 150 000 gal. per day (160 to 400 liters per sec.). Provision is made for a flow of 13 700 000 gal. per day (600 liters per sec.), beyond which the excess is by-passed.

There are two Geiger screens, 11.8 ft. (3.60 m.) in diameter and

* *Gesundheits-Ingenieur*, December 21st, 1912.

5.25 ft. (1.60 m.) wide. As at Gleiwitz, the free opening is 0.12 in. (3 mm.). Only one screen is operated at a time.

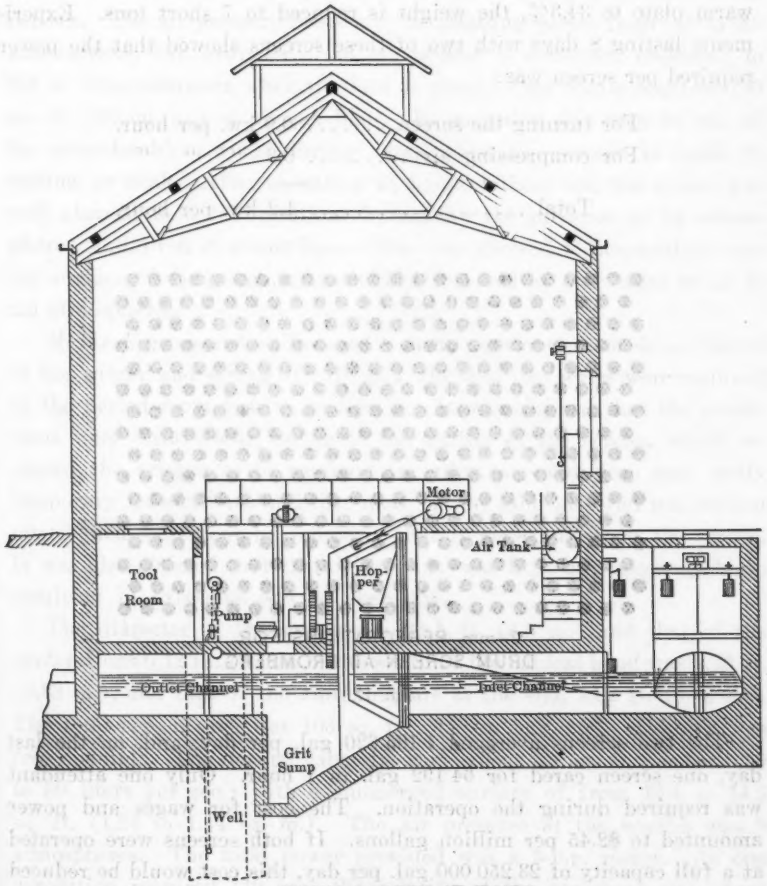
From 4 to 8 cu. yd. (3 to 6 cu. m.) of screenings, containing from 60 to 70% of moisture, are removed daily. With a flow of 4 600 000 gal. per day (200 liters per sec.), the power consumed is 1.8 kw. per hour, but, for the two screens with auxiliaries, 14 h.p. is provided. With one screen in operation only one man is required, and the operating costs are small.

Drum Screens.

Quite different in design from the types mentioned heretofore is the "drum screen" (*Siebtrommel*) first used by Metzger, at Bromberg, and later improved and patented by Windschild, of Dresden-Cossebaude (Fig. 16). In the original form this screen consists of a short cylinder rotating on a horizontal axis, with the curved portion and a ring at the effluent end composed of perforated plates. The sewage enters at one end and passes out through the perforations, which intercept the grosser solids. The interior surface is divided, by radial plates, attached to the end plates, into from 16 to 20 compartments, in which the screenings are raised by the rotation of the drum. They are prevented from falling toward the center by a curved guide-plate concentric with the drum, until near the top, when they drop to a can, hopper, or conveyor, placed in the center. The material tending to clog the perforations is blown inward by an air jet delivered by a swinging nozzle on the exterior, which passes each hole at least twice during the rotation of the screen.

The usual dimensions are: diameter, from 8 to 13 ft.; depth, 1.6 ft.; width of annular end plate, 10 in.; aperture for the air jet, from 0.010 to 0.012 by 1.2 to 2.0 in. As to the perforations in the screen, Metzger found that with a wire mesh a fuzzy mat or film of detritus formed over the surface. He also found that holes 0.4 in. in diameter clogged more readily than smaller ones. Both water and steam were tried for cleaning, but were found objectionable by increasing the moisture in the screenings. Steam, too, resulted in an objectionable odor. Compressed air gave the best results, but, to prevent the dissemination of spray or mist throughout the room, the jet may be protected by a box-like covering.

Bromberg.—At Bromberg there are four sereens. Their diameter is 8.2 ft., and that of the perforations, 0.08 in. (Fig. 17). The screen rotates once in 50 sec., giving a rim velocity of 6¼ in. per sec. The



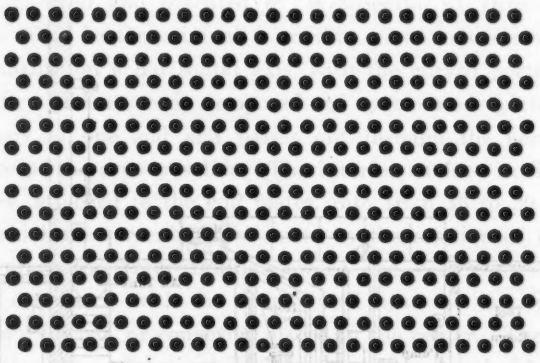
DRUM SCREEN AT TRIER

FIG. 18.

nozzle swings back and forth 85 times per min. With a slit 2 in. long, this requires 1 235 cu. ft. of free air, or from 70 to 77 cu. ft. of compressed air per hour. This has been considered too great, and the design is subject to improvement.

The screenings are removed by a band, 2 ft. wide. This machine screens 1 160 000 U. S. gal. per day, and 4¾ short tons (rising to a maximum of 10¾ tons) of material per million gallons are removed. This material contains from 40 to 60% of moisture. After drying on a warm plate to 34.3%, the weight is reduced to 7 short tons. Experiments lasting 8 days with two of these screens showed that the power required per screen was:

For turning the screen.....	0.9 kw. per hour.
For compressing air.....	0.3 " " "
Total	1.2 kw. per hour.



2–mm. PERFORATIONS FOR
DRUM SCREEN AT BROMBERG

FIG. 17.

The two screens averaged 1 195 620 gal. per day, and, on the last day, one screen cared for 64 192 gal. in 1 hour. Only one attendant was required during the operation. The cost for wages and power amounted to $2.45 per million gallons. If both screens were operated at a full capacity of 23 250 000 gal. per day, this cost would be reduced to $1.09 per million gallons.

The screen-house, with deep foundations, cost $15 600, and the mechanical plant, complete, $7 450.

One of the improvements introduced by Windschild was the use of a truncated cone, instead of a cylinder, for the drum, this form being better adapted to fluctuations of flow. During periods of least flow,

only the conical surface comes in contact with the current, and in wet weather, the sewage passes through both the cone and the annular end plate. Windschild also perfected the form and application of the air jet. A fair velocity of the perimeter of the screen is 4 in. (100 mm.) per sec. The air pressure necessary for cleaning varies from 0.5 to 2.0 atmospheres, but, ordinarily, 0.6 atmosphere is sufficient, increased to 0.8 or 1.0 atmosphere when the load is great. One nozzle requires 700 cu. ft. (20 cu. m.) of air per hour. In operation this may be cut off for considerable periods during times when the sewage is small in volume or weak. After operating 4½ hours without air, the screen was well cleaned in a few rotations by raising the pressure to 1½ atmospheres, and, even if a continuous film was allowed to accumulate over the surface of the screen, this could be removed by a pressure of 1.5 to 2.0 atmospheres.

Mainz Experiments.—Tests of a drum screen were made at Mainz in the winter and spring of 1910-11. The first two series were confined to the period from 8 A. M. to 5 P. M. It was claimed that the conditions were unfavorable, owing to the setting of the screen, which required the sewage to be pumped in order to avoid deep and costly temporary foundations, and that more or less solid material was broken up to such an extent by pumping that it passed through the screen. It was also claimed that the air pressure was kept unnecessarily high, resulting in excessive cost of operation.

The diameter of the screen was 11.5 ft. (3.5 m.) and that of the perforations 0.12 in. (3 mm.); the width of the conical band was 2.73 ft. (0.83 m.), and that of the annular plate at the end, 1.32 ft. (0.52 m.). The total screen area was 103 sq. ft. (9.6 sq. m.), 55% of which was free opening. The normal capacity was 1.75 to 2.8 cu. ft. per sec. (50 to 80 liters per sec.) with a submerged surface of from 13.4 to 14.5 sq. ft. (1.24 to 1.34 sq. m.). The air pressure at the nozzles was 2 atmospheres. The total power provided was a 5-h.p. motor. As one revolution required 110 sec., the velocity of the periphery was about 4 in. per sec.

The power required to operate the drum alone was, in the first experiment, 1.5 to 1.9 kw. per hour, and in the second, 2.0 kw. per hour. For running the compressor, 2.4 kw. per hour were consumed in the first experiment, compressing to 1 atmosphere, and 3.2 kw. per hour in the second, compressing to 1.5 atmospheres.

In the third experiment one-quarter of the drum was covered with brass plates having 0.04 by 0.08-in. (1 by 2-mm.) slots and three-quarters with copper wire with 0.04, 0.06, and 0.08-in. (1, 1½, and 2-mm.) mesh. These results were more favorable as to power required, but indicated the desirability of some alterations of design.

The results in detail are given in Table 5*.

In general, it may be said that, notwithstanding the unfavorable conditions mentioned, and with a sewage containing barely 1 ton of suspended matter per million gallons (0.109 kg. per cu. m.), the screen removed from 46 to 61% of the solids, or from 0.46 to 0.52 cu. yd. per million gallons. The screenings contained about 75% of moisture. The cost ranged from 89 cents to $3.42 per million gallons, decreasing with an increase of flow.

The superintendent of the plant reported the operation of the screen as cleanly and without spattering or objectionable spray or mist.

It was concluded from these tests:

1.—That, on account of the large percentage of free opening, the exposed surfaces are unusually small;

2.—That, in spite of the limited exposed area of the screen, 53.5% of the suspended matter was removed;

3.—That the screen never clogged;

4.—That it did not fail to operate when overburdened;

5.—That no slime or rust spots appeared on the surfaces;

6.—That the cost of power was high, but that this was due in part to unfavorable conditions, and that this drawback would usually be offset by the small cost of installation for a given capacity.

Trier.—A drum screen was installed at Trier (Figs. 15 and 18) in December, 1912, to take the place of tanks. The latter were found costly to operate on account of difficulty in disposing of the sludge. The sanction of the Government to make the change was obtained, with the proviso that particles more than 0.06 in. (3 mm.) in diameter be removed.

The population of Trier is about 40 000. The water supply is 32 gal. (120 liters) per capita, and the sewage flow amounts to 1 270-000 gal. (4 800 cu. m.) daily, half of which runs off in 9 hours, or

* "Siebtrommel mit Druckluftreinigung," by Zivilingenieur G. Windschild.

TABLE 5.—Results of Tests of Windschild's Drum Screen at Mainz, From November 22d to December 3d, 1910.

Date.	Hours in operation.	Volume of Sewage. In cubic feet per second.	Volume of Sewage. In gallons per day.	Screenings. Per Day. Pounds.	Screenings. Per Day. Cubic yards.	Screenings. Per Million Gallons. Pounds.	Screenings. Per Million Gallons. Cubic yards.	Deposit in Grit Chamber. Pounds.	Deposit in Grit Chamber. Cubic yards.	Loss of head, in feet.	Power consumed, in kilowatt-hours per day.	Cost. Power.	Cost. Wages.	Cost. Total.	Cost. Per million gallons.*	Hours compressor was out of service	Remarks.
Nov. 22, 1910.	7	4.94	931 980	1 107.0	0.892	1 185	0.942			.....	31.0	$0.885	$0.583	$1.468	$1.53	....	
" 23, "	8	4.94	1 065 120	1 281.0	0.936	1 202	0.842			.....	31.0	0.885	0.666	1.551	1.44	....	
" 24, "	6	4.94	798 840	638.0	0.425	793	0.495	560	0.254	0.059	26.1	0.617	0.500	1.117	1.35	....	
" 25, "	3	4.94	399 420	488.0	0.313	1 168	0.743			0.131	15.3	0.438	0.250	0.688	1.71	3¼	
" 28, "	8	4.94	1 065 120	702.0	0.576	651	0.495			0.115	26.3	0.752	0.666	1.418	1.26	2⅓	
" 29, "	8	4.94	1 065 120	333.0	0.268	309	0.248			0.049	25.9	0.740	0.666	1.406	1.26	3⅓	
" 30, "	8	4.94	1 065 120	39.7	0.0	33	0.015	384	0.173	0.016	19.7	0.562	0.666	1.228	1.08	4⅓	
Dec. 1, "	7	4.94	931 980	38.6	0.016	33	0.020			0.016	20.0	0.571	0.583	1.154	1.17	2	
" 3, "	6	4.94	798 840	690.0	0.546	860	0.644			0.512	21.8	0.623	0.500	1.123	1.35	....	Much material due to cleaning sewers.
Tests from January 16th to February 1st, 1911.																	
Jan. 16, 1911.	5	2.82	380 500	186	0.089	484	0.198			0.023	16.0	$0.453	$0.416	$0.869	$2.25	1½	
" 17, "	8	2.82	553 000	415	0.173	742	0.297			0.298	33.1	0.945	0.667	1.612	2.88	1	
" 18, "	9	2.82	686 000	640	0.365	927	0.495			0.095	39.9	1.140	0.750	1.890	2.70	....	
" 19, "	8	4.94	1 208 000	.....	.....	.....	.....			.....	.....	.....	.....	.....	.....	.....	
	1	2.82	114 200	772	0.441	675	0.346	850	0.356	0.082	38.6	1.102	0.750	1.852	1.62	¼	Detritus contained much hair, etc.
" 20, "	4	4.94	1 208 000	.....	.....	.....	.....			.....	.....	.....	.....	.....	.....	.....	
	3	2.82	761 000	1 425	0.830	187	1.090			0.095	36.5	1.042	0.583	1.625	2.07	....	Operation suspended after 2.20.
" 21, "	4	2.82	304 500	704	0.427	2 300	1.385			0.089	23.9	0.686	0.333	1.019	3.33	....	Stopped for lack of load.
" 24, "	5	2.82	380 500	459	0.284	1 200	0.742			0.049	18.1	0.516	0.416	0.932	2.43	....	
" 25, "	9	2.82	685 500	684	0.363	993	0.495			0.292	34.7	0.990	0.750	1.740	2.52	2¼	Interrupted from 8.50-9.80 for repairs to motor.
" 26, "	9	2.82	685 500	581	0.293	843	0.396			0.177	34.9	0.995	0.750	1.745	2.88	2½	
" 27, "	4	2.12	228 000	243	0.140	1 059	0.598			0.112	15.8	0.452	0.333	0.785	3.42	1¼	Stopped in P. M. on account of excessive friction.
" 28, "	9	4.94	1 208 000	1 104	0.678	918	0.544	490	0.368	0.272	40.4	1.154	0.750	1.904	1.58	½	Friction increases.
" 30, "	2	4.94	1 208 000	.....	.....	.....	.....			.....	.....	.....	.....	.....	.....	.....	
	7	7.94	1 763 000	1 954	1.159	1 101	0.693			0.804	47.5	1.357	0.750	2.107	1.17	....	Detritus becomes slimy.
" 31, "	6½	7.94	1 392 000	1 422	0.886	1 018	0.593			0.604	35.2	1.004	0.543	1.547	1.08	....	
Feb. 1, "	9	7.94	1 927 000	2 170	1.238	1 076	0.643			0.590	45.3	1.295	0.750	2.045	0.99	....	

* Add for lubricants and polish 4½ cents per million gallons.

TABLE 5.—(*Continued.*)

AVERAGES FOR FLOWS OF FROM 2.12 TO 7.94 CU. FT. PER SEC.

Date.	Hours in operation.	Volume of Sewage.		Screenings.				Deposit in Grit Chamber.		Loss of head, in feet.	Power consumed, in kilowatt-hours per day.	Cost.				Hours compressor out of service.	Remarks.
				Per Day.		Per million gallons.						Power.	Wages.	Total.	Per million gallons.		
		Cubic feet per second.	In gallons per day.	Pounds.	Cubic yards.	Pounds.	Cubic yards.	Pounds.	Cubic yards.								
Nov. 22 to Dec. 3, 1910 and	4	2.12	228 000	232	0.140	1 058	0.594	.....	.....	0.111	15.8	$0.452	$0.333	$0.785	$3.42	....	
Jan. 16 to Feb. 1, 1911	45	2.82	685 500	2 960	1.565	876	0.445	.....	.....	0.154	176.7	5.042	3.750	8.792	2.52	....	
	61	4.94	1 208 000	5 300	3.990	650	0.445	.....	.....	0.098	217.1	6.202	5.080	11.282	1.35	....	
	15½	7.94	1 763 000	3 510	2.125	1 050	0.594	.....	.....	0.597	80.5	2.300	1.292	3.592	1.08	....	

TESTS FROM APRIL 20TH TO APRIL 25TH, 1911.

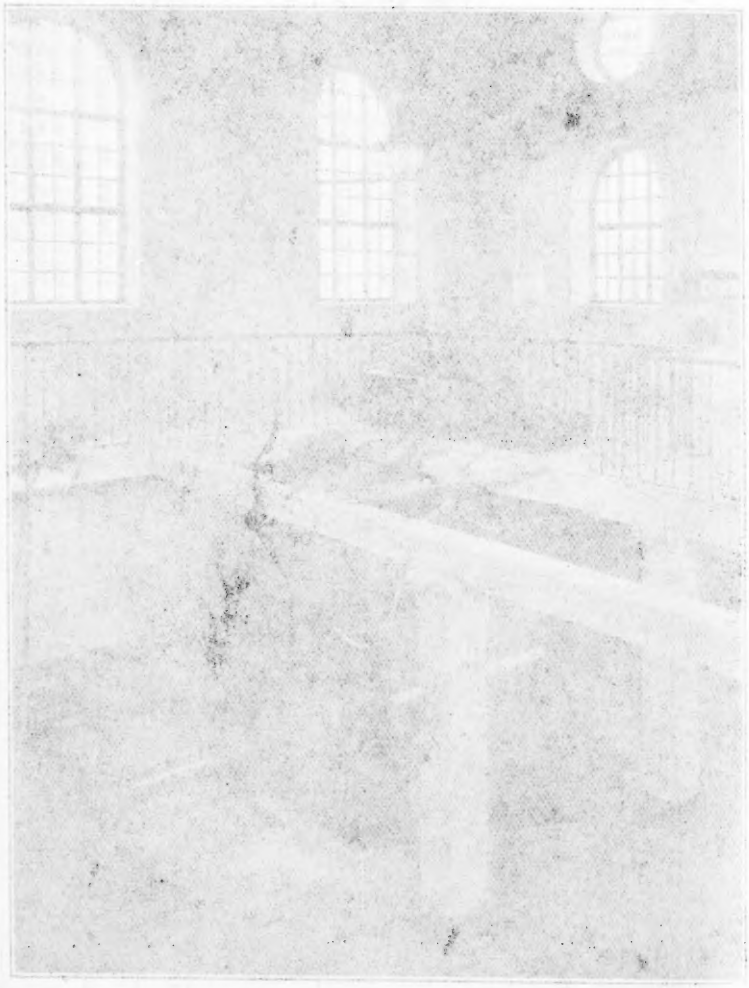
April 20, 1911	7	2.82	685 500	992	0.590	1 860	1.088	4.06	0.202	0.046	19.8	$0.568	$0.583	$1.151	$2.07	1¼	*
" 21, "	10	4.94	1 208 000	2 355	1.312	1 768	0.990			0.174	36.2	1.033	0.833	1.866	1.35	3/2	
" 22, "	7	7.94	1 763 000	2 435	1.317	1 593	0.847			0.830	27.4	0.783	0.583	1.366	0.89	7/8	
" 24, "	10	2.82	685 500	1 644	0.881	2 160	1.138			0.043	36.3	1.037	0.833	1.870	2.43	1/2	
" 25, "	4	4.94	1 208 000	1 274	0.587	2 384	1.188			0.390	16.6	0.474	0.333	0.807	1.44	1/6	
" 25, "	6	2.82	685 500	1 405	0.789	3 069	1.682			0.239	21.8	0.624	0.499	1.123	2.43	1/2	

*The screen was covered: ¼ with brass plate, openings 2 by 1 mm.; ¼ with copper wire 1-mm. mesh; ¼ with copper wire 1½-mm. mesh; ¼ with copper wire 2-mm. mesh.

AVERAGES FOR FLOWS OF FROM 2.82 TO 7.94 CU. FT. PER SEC.

April 20, 1911 to April 25, 1911	23	2.82	685 500	4 040	2.260	2 300	1.286	.....	.....	0.108	77.9	$2.225	$1.915	$4.140	$2.34	....	
	14	4.94	1 208 000	3 690	1.957	2 943	1.088	.....	.....	0.282	52.8	1.509	1.166	2.675	1.35	....	
	7	7.94	1 763 000	2 435	1.317	1 592	0.847	.....	.....	0.830	27.4	0.783	0.583	1.366	0.89	....	

FIG. 18.—DRUM SCREEN AT TRIER. REAR VIEW.



at a rate of 2.6 cu. ft. (75 liters) per sec. The maximum rate of flow is 28 cu. ft. (800 liters) per sec., as determined by gaugings.

The diameter of the screen is 14.5 ft. (4.40 m.) and its width is 3.94 ft. (1.2 m.). The perforations are 0.10 in. (2½ mm.) in diameter and spaced so that only 0.04 in. (1 mm.) of metal remains between the holes. The screen plate is 0.05 in. (1.2 mm.) thick. The screen rotates once in 3 min.

There are sixteen radial plates which raise the coarser screenings and drop them into a central hopper. The fine material which remains on the screen is blown off by three swinging nozzles, the motion of which covers all portions that have been immersed. From the hopper the screenings are delivered to large perforated buckets from which, after draining, they are emptied.

The screen is operated by a 1-h.p. electric motor. A 2½-h.p. motor operates the compressor, ½ h.p. being required to swing the nozzle arms and an equal amount to circulate the cooling water in the compressor. The air is compressed to 2 atmospheres and delivered to a receiver, from which it is led to the nozzles by a pipe and hose.

The screen removes from 0.39 to 0.42 cu. yd. (300 to 400 liters) of material per day; 2.03 tons (3 650 kg.), or about 5.2 cu. yd. (4 cu. m.), have been removed in 11 hours. As this contains only from 50 to 60% of water, it is not sludgy in character, and can be handled easily. It is sold to market gardeners for 14½ cents per cu. yd.

The operation, according to Engineer T. Schürmann, has been found faultless, and no trouble has been experienced from spray or odor.

The total cost of the plant, including buildings, machinery, connections with the sewer, etc., was about $8 600 (36 000 Marks). The cost of operation, exclusive of interest and sinking fund charges, is given in Table 6.

TABLE 6.—Cost of Screening Plant at Trier.

Power, 4.32 kw-hr., 11 hr. for 365 days = 17 265 kw. @ 0.12½ Mark	=	2 158	Marks	= $513.60
One laborer @4.20 Marks per day	=	1 533	"	= 364.85
One helper @ 2.00 " " "	=	803	"	= 191.10
Lubricating oil, waste, and grease @ 0.40 Mark per day	=	146	"	= 34.75
Miscellaneous		60	"	= 14.80
Total cost		4 700	Marks	= $1 118.60
Deduct value of screenings sold		950	"	= 226.10
Net annual cost		3 750	Marks	= $892.50

The price paid by neighboring farmers for the screenings is 0.80 Mark per cu. m. = 15 cents per cu. yd. At 1 270 000 gal. per day,

the cost of operation, therefore, is $\frac{1\,118.60}{365 \times 1.27} = \$2.41$ per million gallons.

Drum screens have also been installed at Insterberg, Worms, Minden, Wartzen, and Osnabrück.

Osnabrück.—Osnabrück is a town of 83 000 inhabitants, 65 000 of whom are served by sewers. Only 17 000 are provided with water-closets. The normal volume of sewage is 1 600 000 gal. (6 000 cu. m.) per day, about 60% of which is collected by newly laid sewers, on the separate system, and pumped, the remainder being collected by combined sewers and delivered by gravity.

The screen is 16.4 ft. (5 m.), converging to 11.48 ft. (3.5 m.) in diameter, and 4.27 ft. (1.3 m.) deep. The perforations are 0.08 in. (2 mm.) in diameter. The maximum capacity of the screen is 33 cu. ft. (950 liters) per sec., with a temporary increase of water during storms, or the sewage from 90 000 persons.

The drum makes a revolution in from 2½ to 3 min. It is cleaned by two compressed air nozzles sweeping the conical portion and one sweeping the down-stream end, each swinging 75 times per min.

The power required for operation is about as follows:

For the screen	2.5 h.p.
For the compressor	5.0 "
For moving air jets	1.5 "
	9.0 h.p.

This is supplied by an alternating-current motor at 380 volts and 24 amperes.

Under ordinary conditions, the force consists of one man, but, during storms, an additional man is required.

The screenings amount to from 5¼ to 6½ cu. yd. (4 to 5 cu. m.) daily, or from 3¼ to 4 cu. yd. per million gallons of sewage, besides which, 0.65 cu. yd. (0.5 cu. m.) of grit is taken each day from a grit chamber placed behind the screen.

At the time the writer visited the plant, some of the perforations were stopped by wool fiber, but these were readily removed with a jet of water. Otherwise, the cleaning device appeared to operate perfectly, and the authorities are so well pleased that a second screen is to be installed soon.

The cost of the one screen, with grit elevator and motors, was

$3 570 (15 000 Marks), but the entire works, including two sewage pumps, air compressor, motor, a Diesel motor for emergency use, heating plant, an independent water supply, building, shower baths, etc., cost $59 500 (250 000 Marks).

The Weand Screen.—A modification of the drum type of screen is found in that invented by O. M. Weand, and now installed at Reading, Pa.,* Atlanta, Ga., Baltimore, Md., and Brockton, Mass. This screen (Fig. 19) consists of a cylindrical frame having a horizontal axis and covered with a fine wire mesh, which is protected by an outer coarse mesh of No. 12 wire. The sewage enters by and overflows from a channel at one end, and passes through the wire mesh to an outlet channel below. The material retained is worked to the farther end by the rotation of the screen, being carried forward by an interior spiral flange. At the farther end, it is lifted by short radial plates nearly to the top, and then falls to a chute delivering to a belt or trough.

Reading, Pa.—The Reading screen (Fig. 19) is 12 ft. long, 6 ft. in diameter, and is covered with Monel metal mesh having about 36 meshes per in. This fine mesh is protected by a $\frac{3}{8}$-in. screen of No. 12 copper wire. It is cleaned by $\frac{1}{16}$-in. water jets, 8 in. apart, from a pipe which swings back and forth longitudinally near the outer surface. Screened sewage instead of water was found to answer very well for this purpose, but crude sewage caused frequent stoppages. The screen rotates about 8 times per min.

The sewage flow amounts to 5 000 000 or 6 000 000 gal. per day, but the capacity of the screen was stated by Mr. Weand to be not less than 8 000 000 gal. per day.

Mr. Kuichling estimates the dried matter removed at 750 lb. per million gallons, or 90 parts per million out of 215 in the raw sewage, corresponding to an efficiency of 42 per cent.

The screenings amount to about 1 cu. yd. (20 to 35 cu. ft.) per million gallons, and have been analyzed as follows:†

Moisture	89.5 %
Mineral matter	2.8 "
Volatile "	7.7 "
	100.0 %

* The Reading screen is not now in use.

† "Sewage Sludge," by Elsner, Spillner, and Allen, p. 208.

They contain much paper pulp, and weigh from 65 to 70 lb. per cu. ft. After passing the screen they are transferred by a bucket elevator and worm conveyor to a tank and then drawn off to a centrifuge which reduces the moisture to 73 per cent. The cost of operation was stated by Mr. Weand to be about $1 per million gallons. Within the past year or so, the use of this screen has been discontinued.

Brockton, Mass.—At Brockton, the sewage is from a separate system, and amounts to 1 750 000 gal. per day, or 40 gal. per capita. It is stronger, therefore, than the average sewage of American cities. The screen is similar to that at Reading, but several mechanical details have been improved. No trouble has been experienced from clogging, except by grease, and, to remove this, the screen is stopped for ½ hour every night and washed with caustic soda. Weak points in the design appear to be lack of proper support for the fine mesh, corrosion of the interior spiral plate, and wear in the holes for the bolts securing the frames to which the mesh is attached.

Table 7 indicates the efficiency of the Brockton screen, the figures being averages for 1912.

TABLE 7.—EFFICIENCY OF BROCKTON SCREEN.

	Suspended Solids, in Parts per Million.		Percentage of reduction.
	Sewage.	Effluent.	
Total	983	282	71.3
Organic	897	245	72.7

During 1912, the average weight removed per day was 4 891 lb., or 1.4 tons per million gallons. In the first week of February of that year 4.5 tons, or 8 cu. yd. of screenings were removed from 2 000 000 gal. of sewage.

The screen required the attention of one man, three men being employed in 8-hour shifts.

Baltimore, Md.—Screens of the Weand type have been installed at Baltimore, Md. (Fig. 20), and Atlanta, Ga., primarily to prevent the clogging of the filter nozzles. The efficacy resulting from the Baltimore screen has already been referred to. At Atlanta the effluent

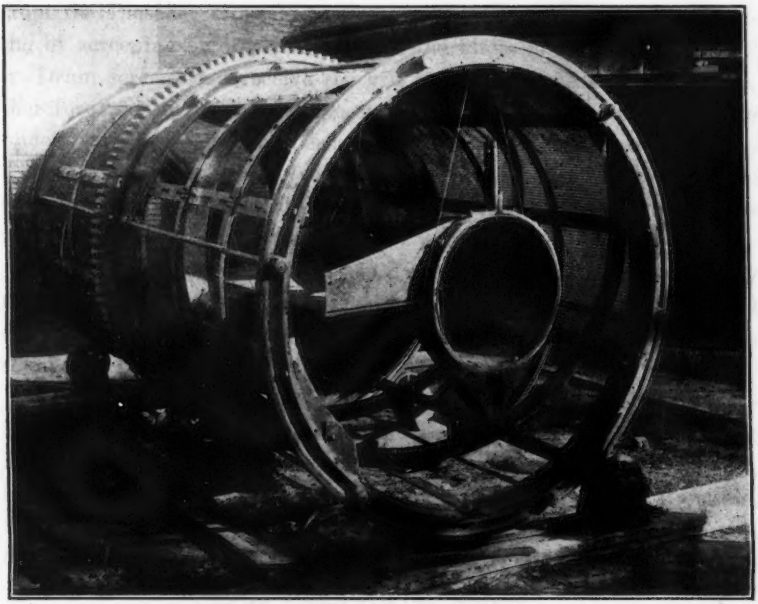


FIG. 19.—WEAND SCREEN PARTLY CONSTRUCTED. INLET END.

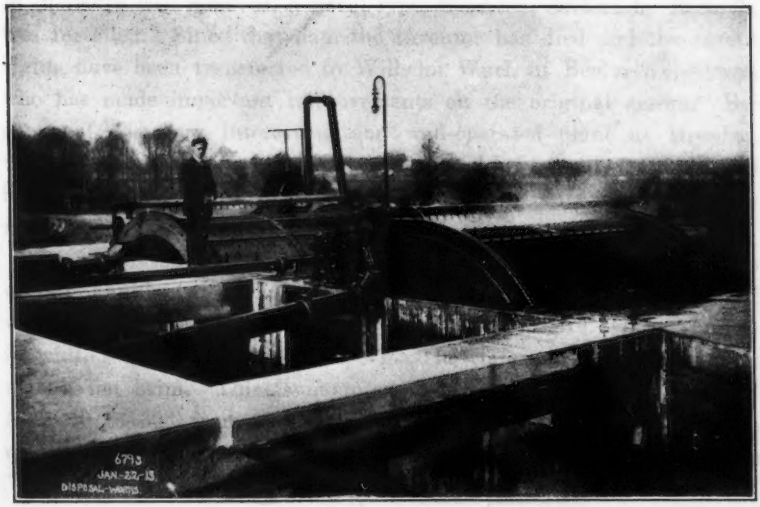
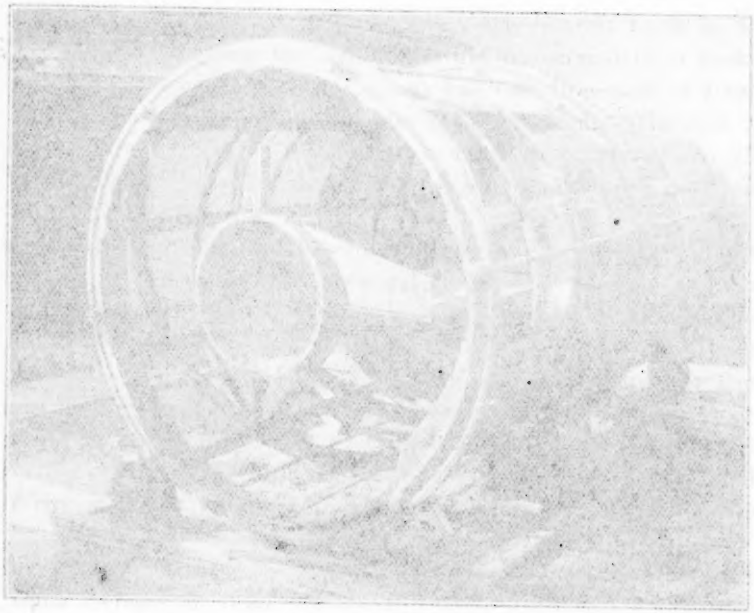


FIG. 20.—WEAND SCREENS IN OPERATION AT BALTIMORE, MD.



from the Emscher tanks has been so satisfactory that further cleaning by screening before application to the filters is not required.

Drum screens can receive the grit as well as true screenings, and, therefore, do away with the cost of grit chambers. They operate under varying loads at speeds that may be adjusted accordingly. They are compact, operate continuously, without hand labor, and the screenings, when cleaned by air, are delivered with a relatively low moisture content.

It is claimed for the Windschild screen, that there is practically no leakage past the outside of the drum, that losses from friction and deterioration are small, and that there is little backing up of the sewage by their use.

In the Weand screen, as installed at Reading and Brockton, there is a loss of several feet of head in the passage through the screen. Friction of the bearings originally gave much trouble at Baltimore, but this has been overcome by recent alterations. At Brockton, the interior flanges and the screen frames have shown considerable deterioration.

The Riensch-Wurl Screen.

Following the permission granted to the City of Dresden in 1903, to discharge sewage into the Elbe, if material greater than 3 mm. in diameter was removed, a novel type of screen, devised by Riensch, was installed. Since that date the inventor has died and the patent rights have been transferred to Wilhelm Wurl, of Berlin-Weissensee, who has made important improvements on the original design. Because of the very interesting and well-operated plant at Dresden, this screen has attracted much attention, and has already been adopted by a number of cities, including Bremen, Christiania, Stettin, Karlsruhe, Mainz, Strassburg, St. Petersburg, Toulon, and Astrachan.

In its present form the Riensch-Wurl screen (Fig. 21) consists of an annular disk surmounted by a truncated cone, both of which are perforated with slots from 0.04 to 0.20 in. (1 to 5 mm.) in width.* In general, the shape may be compared to that of a hat with a flat brim. This is mounted and rotates on a shaft inclined from 10 to 30° from the vertical, and is partly submerged. As a result, the floating solids are raised gently above the surface by the

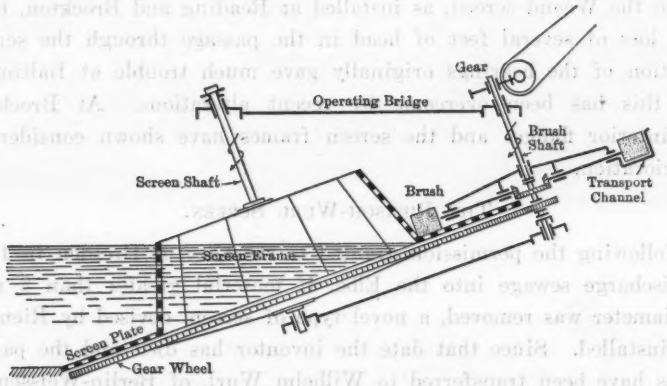
* It is claimed that ½-mm. slots are practicable.

rotation of the screen, and while in this position are brushed off to a container or a conveyor by brushes revolving on arms.

As the channel conforms to the rim of the disk, it is curved in section, and, therefore, there is a smaller variation in level for a given range of discharge than with a rectangular section.

Under ordinary conditions, the cone remains above the surface and so does not require the operation of the special brush provided to clean it, but, as the flow increases, this, too, is brought into service. The speed of rotation can be varied with the flow.

The screen plates are of brass or bronze. The perforations are beveled so that material which has once entered the slot will pass through without clogging.



ELEVATION OF A RIENSCH-WURL SCREEN

FIG. 21.

The brushes are of wild hogs' bristles or piassava fiber. They are cylindrical in form, and adjustable by counterweights so as to press but slightly on the screen. During operation, they rotate on their axes, so that the material is not pressed down or even scraped forward, but is rolled before the brushes until it falls on the conveyor or into the receptacle provided for it. By this means there appears to be no clogging of the holes except by a gradual deposit of grease, which is removed every few months by a jet of dry steam. The brushes themselves are cleaned by passing over a comb at each revolution. The speed and arrangement of brushes is such that each part of the screen is cleaned four or five times before being again submerged.

A "service bridge" is constructed across and above the screen, to which are attached the ball-bearings on which the screen frame is hung. There is only one bearing that is submerged, and this serves as a guide for the lower end of the shaft. It is protected by a stuffing-box lubricated by oil under pressure, and no trouble has been experienced as yet from the introduction of grit or other foreign material. The ball-bearings run in oil, but slow-moving parts are lubricated by grease.

One feature peculiar to the Riensch-Wurl screen is the large percentage of surface available for actual use.* The approximate area exposed to the sewage, the capacity and power required for screens of different sizes but with slots 2 mm. wide, are as given in Table 8.

TABLE 8.—SIZES, CAPACITIES, ETC., OF RIENSCH-WURL SCREENS.

Diameter, in feet.	Exposed area, in square feet.	Capacity, in gallons per day.	Horse-power required.
6	25	1 800 000	¼
12	90	4 200 000	
14	115	7 200 000	½
20	250	28 800 000	
26	405	103 000 000	2½

Screens of this type, with 2-mm. slots, will ordinarily remove from 30 to 40% of the suspended solids, and, with 1-mm. slots, sometimes from 50 to 60%, but the exact quantity depends on the character of the sewage. Under favorable conditions these percentages may be increased by one-half.

Ordinarily, the head lost in passing the screen is from ¾ in. to 4 in. (2 to 10 cm.), but this reaches 10 in. (25 cm.) or more during periods of high flow. It is estimated approximately as follows by Mr. Wurl: It is assumed that the effective opening is equal to about 25% of the submerged surface under any given rate of flow, and that from one-half to one-third of this is covered with screenings and, therefore, useless.

Let S = the submerged surface, in square feet;

A = the true effective area = say, $0.25 \times 0.4\ S$, surface = $0.1\ S$;

Q = the quantity of sewage, in cubic feet per second;

* Claimed by the manufacturer to be 80%, as compared with 33⅓% for band screens and 20% for wing screens.

$G = 32.2$;

C = a coefficient provisionally assumed as 0.4.

Then, the head lost, in feet, is:

$$H = \left(\frac{Q}{A\,C\,\sqrt{2\,G}}\right)^2 = \frac{Q^2}{10.3\,A^2} = 0.097\,\frac{Q^2}{A^2} = 9.7\,\frac{Q^2}{S^2}.$$

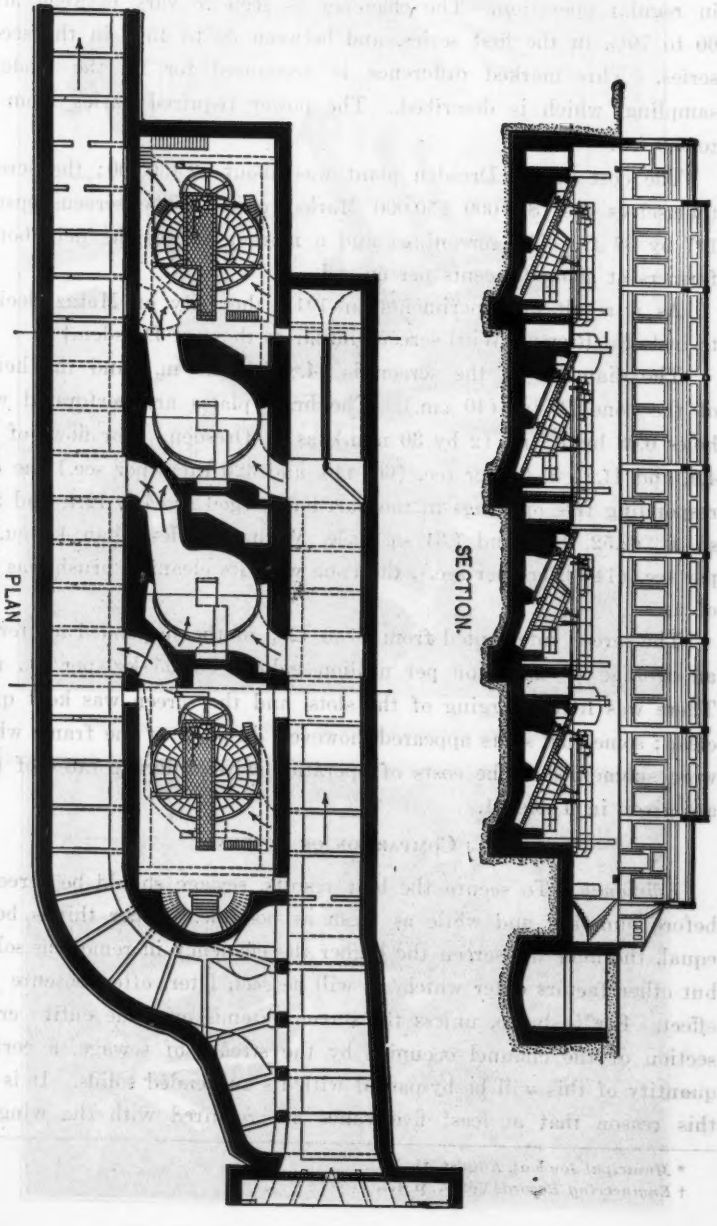
Dresden.—The Dresden plant (Figs. 22 and 23) consists of a grit chamber, a 2.65-in. (66-mm.) fixed-bar screen, and four Riensch-Wurl screens, the first of which was installed in 1910. The population served is about 530 000, but, of these, 280 000 had water-closet connections in 1913. The average sewage flow is 26 500 000 gal., including industrial wastes, or 50 gal. per capita daily, but this varies from 18 to 21 cu. ft. (500 to 600 liters) per sec. between 3 and 6 A. M. to about 70 cu. ft. (2 000 liters) per sec. between 11 A. M. and 5 P. M. This flow is easily taken care of by one screen, the other three being held in reserve for use in storms, when the flow increases at times to from 175 to 260 cu. ft. (5 000 to 7 500 liters) per sec.

Each of the screens (Fig. 24) is 26.2 ft. (8 m.) in diameter, of plate bronze 0.2 in. (5 mm.) thick, and contains 230 000 slots 0.08 by 1.20 in. (2 by 30 mm.) (Fig. 25). The total screening surface is about 600 sq. ft. (56 sq. m.). During ordinary conditions, the screen rotates once in about 3 min., but during storms this rate may be increased 50 per cent. About once in 3 or 4 months it is found necessary to clean the slots by steam. The head required to pass the screen varies from 2 to 12 in. (5 to 30 cm.). It averages about 4 in. (10 cm.), and often reaches 6 in. (15 cm.).

After passing the screens, the effluent is pumped with a lift of 23 ft. (7 m.) during high water, to the Elbe, where it is discharged by a 4-ft. (1.2-m.) pipe at a depth of about 6 ft.

An average of 25.6 cu. yd. (19.7 cu. m.) of screenings, or 0.97 cu. yd. per million gallons, or 0.09 cu. yd. per 1 000 inhabitants contributing, were removed daily in 1913, and contained 84% of moisture. This material is placed in a dump where it drains to about 75% moisture before removal by farmers, who pay 4½ cents per cu. yd. (0.25 mark per cu. m.) for it. The odors about the screening dump are quite offensive.

Tables 9 and 10, which are summaries for the two years beginning May 1st, 1912, and March 1st, 1913, give the results obtained



DRESDEN SCREENING PLANT

Fig. 22.

in regular operation. The efficiency is seen to vary between about 60 to 70% in the first series, and between 30 to 45% in the second series. This marked difference is accounted for by the mode of sampling, which is described. The power required varies from 2.5 to 3.0 kw.

The cost of the Dresden plant was about $2 150 000; the screens themselves cost $12 000 (50 000 Marks) each.* The screen-house is 197 by 37 ft. The screenings find a market among the neighboring farmers at about 9½ cents per cu. yd.

As a result of experiments in 1911, the City of Mainz decided to install a Riensch-Wurl screen similar to those at Dresden.†

The diameter of the screen is 14.8 ft. (4.5 m.) and the height of the cone 16 in. (40 cm.). The brass plates are perforated with holes 0.08 by 1.2 in. (2 by 30 mm.), as at Dresden. For flows of 2.1, 4.9, and 11.2 cu. ft. per sec. (60, 140, and 320 liters per sec.) the corresponding free openings in the part submerged are 7.2, 12.4, and 21.2 sq. ft. (2.52, 4.29, and 7.31 sq. m.). With rates less than 4.9 cu. ft. per sec. (140 liters per sec.), the cone with its cleaning brush was out of use.

The screen intercepted from 36 to 42% of the suspended matter, or an average of 0.395 ton per million gallons (0.095 kg. per cu. m.). There was little clogging of the slots, and the screen was kept quite clean; some rust spots appeared, however, on parts of the frame which were submerged. The costs of operation with different rates of flow are given in Table 11.

Comparison of Screens.

Efficiency.—To secure the best results, sewage should be screened before pumping and while as fresh as possible. Other things being equal, the finer the screen the higher the efficiency in removing solids, but other factors enter which, as will be seen later, often obscure this effect. For instance, unless the screen extends over the entire cross-section of the channel occupied by the stream of sewage, a certain quantity of this will be by-passed with its suspended solids. It is for this reason that at least five vanes are required with the wing or

* *Municipal Journal*, August 9th, 1911.
† *Engineering Record*, Vol. 67, p. 471.

FIG. 23.—RIENSCH-WURL SCREEN CHAMBER AT DRESDEN.

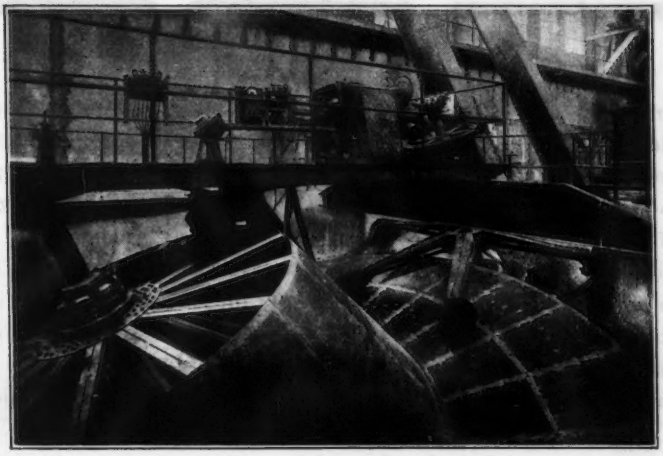
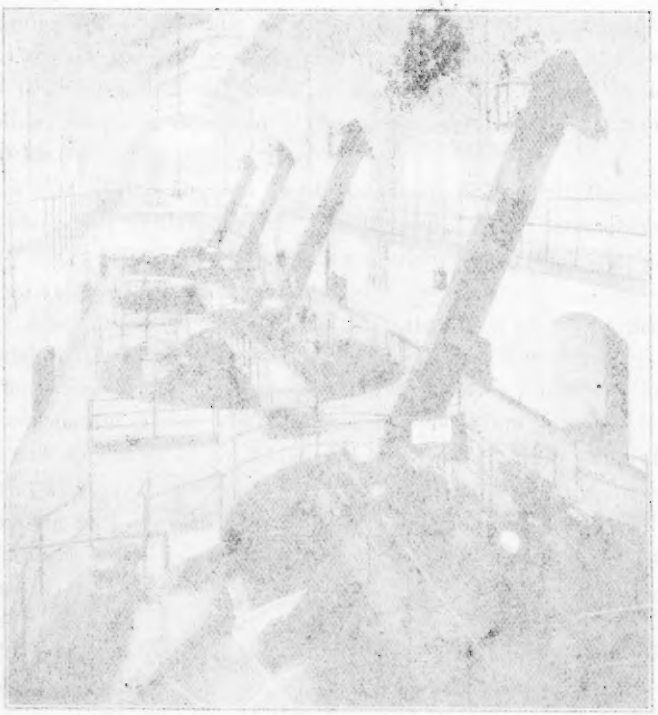
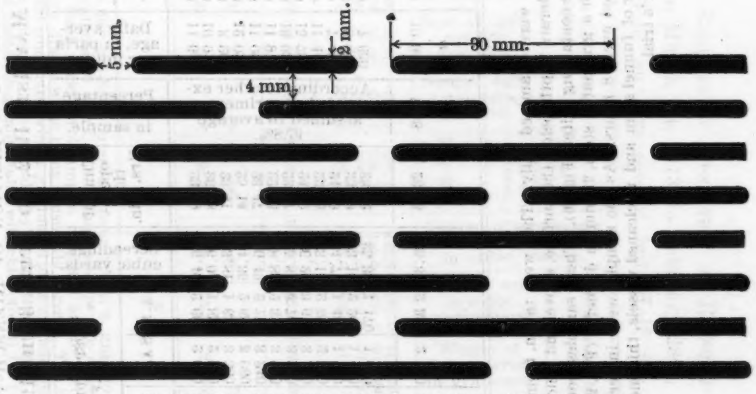


FIG. 24.—RIENSCH-WURL SCREEN AT DRESDEN.



shovel-vane type of screen, and an invert depressed so that every portion of the stream will have to pass through one vane. To insure against leakage past the Riensch-Wurl screen, adjustable plates are set in the channel close to the periphery of the screen plate.

The efficiency also depends on whether the screening surface consists of bars, perforated plates, or a wire mesh, whether the solid particles are broken up by abrasion due to too high velocities or are macerated, so as to be carried through the screen by the current, or whether they are pressed through in the process of cleaning. Velocities through the screen should be moderate, and for this reason it is better that the motion of the screen should not be against the current, and that it should be adjustable.



SLOTS IN RIENSCH-WURL SCREEN AT DRESDEN

FIG. 25.

Round bars are probably undesirable, as they allow floating solids to enter and become wedged in the openings. Bars of a thin rectangular section are preferable as offering less resistance to the flow and having greater strength than other forms for the same quantity of metal. At Hamburg, the links are 0.20 in. (5 mm.) thick and are 0.6 in. (15 mm.) apart; at Frankfort and Elberfeld the bars are 0.12 in. (3 mm.) thick and are 0.4 in. (10 mm.) apart. Experiments made on wooden strips, 3 in. by ½ in. in section, at Cornell University* indicate a co-efficient of discharge of 0.811 if the sides are left parallel, increasing to 0.832 if sharpened on the down-stream edge, and becom-

* *Engineering Record*, Vol. 62, p. 762.

TABLE 9.—RESULTS OBTAINED BY OPERATION OF RIENSCH-WURL SCREENS AT DRESDEN, FROM MAY 1ST, 1912, TO APRIL 30TH, 1913.

Month.	Suspended Matter in Sample Before Screening.						Daily average, in parts per million.	Percentage of moisture in sample.	Time of operation, Hrs. Min.	Screenings, cubic yards.	Suspended Matter in Sample After Screening.						Daily average, in parts per million.	Percentage of moisture in sample.	Percentage of efficiency of screen.
	4 A. M.	8 A. M.	12 M.	4 P. M.	8 P. M.	12 MN.					4 A. M.	8 A. M.	12 M.	4 P. M.	8 P. M.	12 MN.			
May.....	9 800	9 080	11 020	12 690	13 090	14 790	11 640	According to other extended experiments, assumed to average 97.8%.	23 29	26.40	2 660	2 620	3 760	3 770	3 470	5 080	3 560	According to other extended experiments, assumed to average 95.7%.	69.4
June.....	8 520	9 600	11 320	10 890	12 170	12 980	10 910		26 28	26.04	1 830	2 980	3 550	3 580	3 620	3 280	3 140		71.2
July.....	4 580	10 090	12 040	12 040	7 140	5 510	8 260		23 41	23.81	1 630	3 360	4 080	4 090	3 810	2 910	3 310		59.9
Aug.....	9 860	8 950	12 760	13 500	13 880	13 210	12 020		25 14	24.35	2 680	3 060	3 460	4 120	4 140	4 600	3 670		69.4
Sept.....	8 740	7 830	12 270	12 120	13 480	12 040	11 080		23 47	26.49	2 770	3 240	4 380	4 870	4 320	4 520	4 010		63.8
Oct.....	9 030	9 210	12 270	14 410	13 990	12 940	11 970		22 36	28.88	2 310	3 930	5 310	5 110	4 450	3 420	4 090		65.8
Nov.....	11 140	12 400	17 010	15 150	14 620	13 270	13 930		23 17	25.81	3 700	3 270	4 600	4 780	5 000	4 810	4 360		68.7
Dec.....	11 990	11 290	18 440	21 060	18 000	13 910	15 780		24 09	26.26	3 610	3 220	5 810	6 300	4 870	4 490	4 710		70.1
Jan.....	9 770	7 280	11 160	13 140	13 600	13 700	11 440		22 45	23.15	2 520	2 100	3 880	4 470	4 350	3 550	3 470		69.6
Feb.....	4 220	5 290	9 350	10 620	8 130	6 900	7 420		24 39	24.71	1 480	1 900	4 190	4 220	3 590	3 380	3 130		57.8
Mar.....	7 390	4 840	9 890	9 260	8 690	8 280	8 060		23 19	23.15	2 200	1 430	4 280	4 260	4 040	3 110	3 220		60.0
Apr.....	4 260	4 990	9 300	10 180	8 970	8 200	7 650		23 12	25.32	1 170	1 690	4 650	3 680	3 740	3 390	3 050		60.1
	8 270	8 400	12 230	12 720	12 140	11 310	10 840 *a*	97.8	23 53	25.35	2 380	2 730	4 330	4 440	4 110	3 880	3 640 *b*	95.7	66.4 $\frac{a-b}{a}$ =

FIG. 26.

Six samples were examined daily. They were taken, from 4 A. M. to midnight, from a moderate depth below the surface, above and below the movable screen, by buckets containing 1 liter (Fig. 26). These samples were decanted into a glass funnel with a graduated stem, 21 mm. in diameter (Fig. 27). The settled sludge was removed every 8 hours. As the samples were imperfect, because of the small diameter of funnel stem and uncleaned vessels, this method was abandoned after a year's trial.

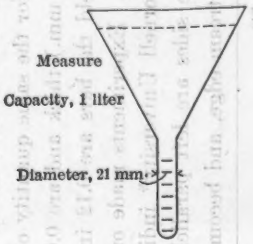


FIG. 27.

TABLE 10.—Summary of Results of Operation of Riensch-Wurl Screens at Dresden, From March 1st, 1913, to February 28th, 1914.

Note: Fecal Matter from an Average of but Half the Properties Connected.

Month.	Suspended Matter in 10-Liter Samples of Sewage Before Screening.						Time of operation of screen.	Volume of screenings removed, in cubic meters.	Suspended Matter in 10-Liter Samples of Sewage After Screening.						Percentage of Efficiency of Screen.	
	Volume wet, in cubic centimeters.	Weight of dry material, in grammes.	Loss by incineration, in grammes.	Residue after incineration, in grammes.	Sum of daily specific gravities.	Sum of daily moisture, per cent.	Hrs. Min.		Volume wet, in cubic centimeters.	Weight of dry material, in grammes.	Loss by incineration, in grammes.	Residue after incineration, in grammes.	Sum of daily specific gravities.	Sum of daily moisture, per cent.	By volume.	By weight.
Mar., Apr., May	1 153.0	50.9621	31.2458	19.7163	23.57	1 535.9	544 10	442.9	782.0	41.4080	24.4710	16.9320	23.67	1 526.6	32.17	18.75
June, July, Aug.	968.0	61.4275	33.6803	27.7472	33.08	1 915.1	550 15	415.3	663.5	50.7896	26.3830	24.4066	33.51	2 000.1	31.45	21.25
Sept., Oct., Nov.	1 257.0	81.7181	47.2796	34.4385	37.41	2 204.0	566 05	484.5	902.0	59.1381	33.6270	25.5111	37.16	2 206.5	28.24	27.63
Dec., Jan., Feb..	1 193.5	99.4181	55.6602	43.7579	30.84	1 809.8	548 30	435.3	686.0	51.3313	26.9992	24.3321	30.27	1 717.6	42.52	45.50
Total.....	4 571.5	293.5258	167.8659	125.6599	124.90	7 464.8	2 209 00	1 778.0	3 033.5	202.6620	111.4802	91.1818	124.61	7 450.8		
Equivalent days of examination	85	79	79	79	77	78	90	90	85	79	79	79	77	78	$\frac{a-b}{a}$ =	$\frac{c-d}{c}$ =
Average.....	53.78	3.7155	2.1248	1.5906	1.62	95.7	24 33	19.7	35.68	2.5653	1.4111	1.1542	1.62	95.5	33.64	30.95
		a	*c*							*b*	*d*				As to organic material as shown by loss on incineration.....	33.59

Six samples were taken every fourth dry day at 4-hour intervals, from 2:30 A. M. until 10:30 P. M., at designated points above and below the screen by a "Hofer" apparatus (Fig. 28) containing 20 liters. The samples below the screen were taken on the average about 20 sec. later than those above, depending on the velocity. Each series of 6 samples collected at the two points was placed in a vessel holding 120 liters. From these vessels 10 samples of 1 liter each were drawn while being constantly stirred. The latter average daily samples from above and below the screen were poured into a tin funnel (Fig. 29) connected with a graduated glass cylinder 35 mm. in diameter and having a capacity of 150 cu. cm. All the sludge deposited within 4 hours was collected in the cylinder by jarring the funnel. It was then removed and the contents of the cylinder sent to the laboratory (after being rendered bacteriologically stable with chloroform) for further examination.

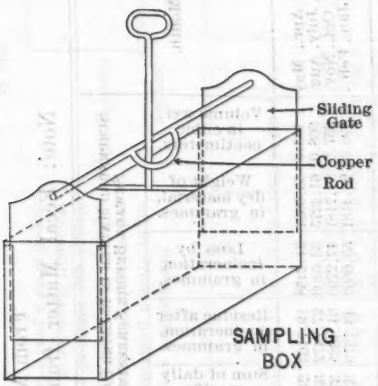


FIG. 28.

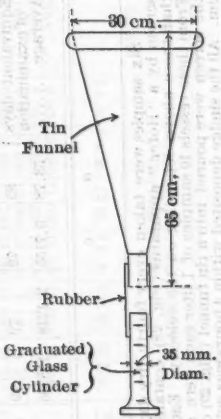


FIG. 29.

ing 1.032 if sharpened on both edges. The great advantage in the latter cannot be availed of, for the same reason that round bars are objectionable. In the shovel-vane screen at Gleiwitz, the curved bars taper on the down-stream side or toward the center of the screen. Aside from the small hydraulic advantage, this permits material once passing the up-stream surface of the vane to pass through freely.

TABLE 11.—COST OF OPERATION OF SCREENS AT MAINZ, WITH DIFFERENT RATES OF FLOW.

RATE OF FLOW:		COST:	
In cubic feet per second.	In liters per second.	In dollars per million gallons.	In pfennigs per cubic meter.
2.1	60	$1.76	0.195
2.8	80	1.32	0.145
4.9	140	0.74	0.082
6.3	180	0.58	0.064
8.4	240	0.43½	0.048
11.2	320	0.32½	0.036

Perforated plates, as in the Riensch-Wurl and Windschild screens, have an advantage in intercepting straws, matches, hair, and fiber which might pass between bars or even slots with equal spacing.

The American wire-mesh screens of Weand and Jennings are the finest of all, each mesh, including the wire, being only about 0.03 in. (0.75 mm.) square. Objection has been found with mesh screens in

that they promote the disintegration of the solids, so that an appreciable quantity passes through with the liquids. This was observed by Watson in connection with a Carshalton screen at Birmingham, and in connection with the Riensch-Wurl screen, and has also been mentioned with reference to tests of the shovel-vane screen at Gleiwitz.

A very important feature in any screening device is the method of cleaning, as the thoroughness of this operation directly affects, not only the efficiency, but also the cleanliness of the plant and the reliability of operation. There are many ways of cleaning bar screens. In some, particularly in England, the fixed bars are cleaned by movable rakes, but as it is preferable that all cleaning be done above water, in order to prevent the escape of broken-up particles with the effluent, it follows that the screen itself should raise the detritus to the proper elevation for removal. This is a feature of all the fine screens which have been described in detail herein.

The Hamburg screen is cleaned automatically by a hard-rubber rake or comb which engages the links after they have passed over the top, and is then withdrawn past a scraper which causes the screenings to fall on a belt conveyor. The motion of both rake and scraper is transmitted from a cam on the driving shaft by a system of levers. The device is carefully designed and appears to work satisfactorily.

The Jennings screen is cleaned by a jet of compressed air. Where grease collects in spite of this, it may be removed by the substitution of steam for a short period. The screenings are then removed by a conveyor. So far as known, this screen has not yet been tested out with domestic sewage.

Wing screens are generally cleaned automatically by a scraper which may, or may not, be followed by a brush. At Elberfeld it is found necessary to clean out the spaces by hand about once a week. This takes about ½ hour, but does not interrupt operation. At Bradford, England, where the sewage contains considerable wool fiber, the Engineer, Mr. Joseph Garfield, has found, after much experimentation, that this is most readily removed by a rake which operates on each vane when brought to and held in a horizontal position.

The shovel-vane and drum screens have an advantage in the removal of the screenings chiefly by gravity, the residue only being removed by a brush or jet.

With the Riensch-Wurl screen, the material is brushed off the smooth plates by automatically operated brushes. As already mentioned, it has been claimed that this operation forces some of the material through the slots, but to the writer the removal appeared to be very complete. Once in 3 or 4 months, however, sufficient grease is deposited in the slots to necessitate its removal by a jet of superheated steam.

Metzger experimented with various methods of cleaning perforated drum screens, and concluded that compressed air was most efficient and inoffensive. It has the advantage of blowing the detritus into the drum and delivering it with less moisture than, for instance, occurs in screenings from the Weand screen, where the mesh is cleaned by moving jets of water. Screenings from the Weand screen contain nearly 90% of moisture, but in those from the Windschild screen the quantity is only two-thirds as great. The reason for so much moisture in the Jennings screenings is not apparent. A further objection to cleaning by water jets is the disintegration of the solid material, allowing it to pass the screen.

The thoroughness of screening is best indicated by the parts per million of dried suspended matter removed from identical sewages. Some valuable data of this kind were contained in a paper by Emil Kuichling, M. Am. Soc. C. E., read before the New Jersey Sanitary Association in December, 1908, in which the material removed by grit chambers and screens is given in pounds of dry matter per million gallons of sewage, but such information is not often obtainable from the reported results of screening. If one cannot find the parts per million of suspended matter before and after screening, the next best information is the cubic yards (or tons) of screenings secured per 1 000 population, or per million gallons of sewage, and the percentage of contained moisture. A record of this, kept at every fine-screening plant, would not require great effort and would furnish most desirable information regarding the efficiency of the screen and of its operation. It should be borne in mind that the weight of wet screenings is materially reduced by a brief period of draining.

Table 12 contains general data of this nature for the several types of screens under consideration, which the writer has collected.

The results stated in Table 12 do not indicate the increased efficiency due to fineness of screen that one would naturally look for. More

good experiments with different types of screens, and under identical conditions, are needed, such as those attempted in the comparative tests of the Windschild and Riensch-Wurl screens at Mainz. In general, the data collected show that from ½ to 1½ cu. yd. or more of screenings may be removed from 1 000 000 gal. of ordinary domestic sewage at a cost of from $1 to $2, depending on the conditions. Further experience will probably develop the fact that care in operation has much to do with the station efficiency of any fine screen. A case has been brought to the writer's attention where a change of administration was the apparent cause of a decrease of 16% in the annual efficiency. The importance of using the most approved method of sampling has already been indicated with regard to the results obtained at Dresden.

It has been clearly demonstrated through experiments at Ithaca by Messrs. T. C. Schaetzle and E. R. Davis* with a ½-in. wire mesh screen, that the efficiency varies greatly with the strength of the sewage. At 4 A. M., 2.6 lb. of dry material were removed per million gallons, and at 11 A. M., 104 lb., with an average for the 24 hours of 52.7 lb.

To obtain true average efficiencies, the hourly efficiencies should be weighted according to the rate of flow, in a way similar to that by which composite samples of average water or sewage are prepared for analysis. This is a point, by the way, which appears to be generally neglected.

The influence of pumping and of length of travel, in comminuting the solids and increasing the colloidal matter, should be kept in mind as factors which may cause a marked lowering of efficiency.

The effect of preliminary cleansing by coarse screens or grit chambers should properly be considered in passing judgment on the relative merits of different methods of fine screening.

Adequacy and Reliability.—All the fine screens which have been described depend on continuous motive power for operation, and, having this, they appear to be quite dependable within the ranges of sewage level for which they are designed. In the case of the band screen, the surface may rise nearly to the belt conveyor, which is usually just below the top; with the wing, shovel-vane, and drum screen, it may rise to a point just below the axle, and, with the Riensch-Wurl screen, to the highest point reached by the base of the cone during a revolution. Larger proportions of band and Riensch-Wurl screens

* *The Cornell Civil Engineer*, November, 1913.

TABLE 12.—DATA RELATING TO THE SEVERAL

Type of screen.	Name of inventor or manufacturer.	Location.	Clear opening, in inches.	Screenings. Per million gallons.	Screenings. Per 1 000 population daily.
Band.....	Brunotte.....	Hamburg....	0.6	0.34 cu. yd.	0.018 cu. yd.
	Hertzberg.....	Göttingen....	0.4	3.8 "	0.282 "
	John Smith & Co..	Sutton.....	0.375 meshes per inch.	0.6 ton	.....
	Jennings	Chicago Stock Yards....	.....	2.4–3.1 tons	.....
Wing.....	J. S. Fries Sohn....	Frankfort ...	0.40	2 "	0.112 ton.
	" "	Elberfeld....	0.40	1.15 cu. yd.	0.053 cu. yd.
		Stralsund....	0.20	.....	0.079 "
		Wiesbaden ..	0.60	1.1 "	0.083 "
Shovel-vane...	Geiger Mach. Wks.	Strassburg ..	0.10	1.6 "	0.043 "
	" "	Gleiwitz.....	0.12	.....	0.192 "
	" "	Temesvar....	0.12	0.9–1.7 "	0.067–0.133 "
Drum.....	Windschild.....	Bromberg ...	0.08	4¾ tons	.....
	"	Mainz.....	0.12	0.52 cu. yd.	.....
	"	Trier	0.04–0.08 / 0.10	0.39–0.42 "	0.13 cu. yd.
	"	Osnabrück...	0.08	3.2 –4.0 "	0.08–0.10 "
	Weand	Reading, Pa.....	(36 meshes per inch.)	1.0 "	.....
	"	Brockton, Mass.....		1.4 tons	.....
Riensch-Wurl..	Riensch-Wurl.....	Dresden.....	0.20	0.97 "	0.09 "

* The figures in this column should be used with caution. More complete data should screens.

therefore are submerged during periods of maximum flow and less head room is required. As the range of level with a Riensch-Wurl screen is limited by the diameter and inclination, this type is not well suited to small volumes of flow combined with wide variations in level. Where subject to these conditions, the band screen offers advantages, as it may be of any length without appreciably affecting other dimensions or costs.

Operation should be automatic, as far as possible. It is nearly so in all these screens, except when the speed is to be varied or the screen is to receive a special cleaning. With speeds that are too low, clogging and backing up of the sewage may result. The fine deposit of grease

SCREENS DESCRIBED IN THIS PAPER.

Percentage of moisture.*	Percentage of efficiency.	Horse power per screen.	Cost of Operation. Per million gallons.	Cost of Operation. Per cubic yard of screenings.	Authorities.	Remarks.
87	.....	2.5	.....	.....	Baudirektor Sperber.	After removal of half this volume of grit.
.....	90 (?)	2.0	.....	.....	E. Kuichling.	
.....	.....	.....	.....	.....	" "	
79	63	.....	.....	.....	C. A. Jennings.	
.....	10	5.0	.....	$0.18	Stadtbaumeister Schaefer.	After removal of 16% by grit chamber.
75	.....	.....	.....	.....	Dipl.-Ing. Sturmfels.	Including 0.6 cu. yd. grit per million gallons.
.....	.....	4.5	.....	.....	Frühling.	
.....	.....	Hand power	$1.64†	.....	Engineer Vogel.	After passing 1.6-in. bar screen.
89.3	10–12	3.35	.....	0.054	Tiefbauamt Strohl.	After removal of 0.132 cu. yd. grit and coarse screenings per 1 000 population.
.....	63	.....	0.90	0.125	Magistrat ——.	
60–70	.....	.....	Small	.....	Stadt-Ing. Vidrighin.	
40–60	.....	.....	2.45	.....	Stadtbauinspektor M. Knauff.	Experimental.
75	53.5	5.2–6.8	0.89–3.42	.....		
50–60	.....	.....	2.41	.....	Abt. Baum. Schurmann, G. Windschild.	" "
.....	.....	9.00	.....	.....	Dipl.-Ing. Schlüssel G. Windschild.	Before removal of 0.4 cu. yd. grit per million gallons.
89.5	42	2.0	1.00 ±	.....	E. Kuichling. O. M. Weand, E. B. Ulrich.	
.....	71.3	.....	.....	.....	C. R. Felton, I. Hayleston.	
84	33.6	2.5	0.325–1.76	.....	Stadtbaurat Fleck, W. Wurl.	

be secured, in order to furnish a reliable comparison between the efficiencies of different

† Including coarse screening, settling, and subsequent screening.

that usually occurs on any screen will require special removal at times, and, with fine perforations or meshes, a mat of fine fiber or hair may gradually form over the surface; otherwise, the cleaning devices appear to be effective.

Reliability of operation depends, also, on the design and workmanship. The simpler the design the better, especially as this facilitates renewals and repairs. The wing screen and some forms of band screen are simpler perhaps than the others, but serious objections under this heading have been pretty well met in the design of the other types. In reference to this it may be stated that the Carshalton screen is usually operated by the hydraulic power of the sewage itself, and, there-

fore is not dependent on outside sources of motive power. The possibility of operation by hand in case of accident to machinery is a claim made for the wing screen, but might apply as well to the others, if of small size.

Nuisances.—If properly operated and kept clean, there need be no objectionable odors or other offensive conditions in the neighborhood of works using these screens. Inside the station there is usually the smell of fresh sewage, which is not strong nor likely to be carried far. With septic sewage, of course, there is more danger of foul odors. Any spattering or spraying of the sewage caused by high speeds or improper design, especially in the application of air, steam, or water jets, for cleaning, increases the odor.

Another and more probable cause for offense lies in the disposal of the screenings. From observation of a great many plants, the writer is convinced that screenings should either be incinerated, artificially dried, or promptly removed from the works. During warm weather, this should not be delayed more than 24 hours from the time of collection. Offensive conditions may be mitigated by composting with sweepings. This question of disposal is one of much importance, and also one which is frequently neglected.

Accessibility.—All parts of a screen should be accessible, for cleaning, repair, and renewal, with the least interruption to operation.

The Hamburg screen is hung on a shaft so that the lower end may be swung above the surface when required. The rotation of the wing, shovel-vane, and drum screens renders all moving parts accessible, and, with the exception of the bearing at the foot of the shaft, this is true of the Riensch-Wurl screen. It is understood that this objection is to be overcome by suspending the screen from a bearing above. In the case of the Frankfort screen, nearly all the moving parts can be replaced without interruption of the flow.

First Cost.—The cost of a screening plant depends, not only on the screen with its auxiliary machinery, but, to a great extent, on the location, foundations, and size of chamber. Simplicity of design, compactness, and the loss of head resulting from its use are all to be considered; in fact, the last may control the selection. As the cost depends on so many factors, the figures of gross cost, which are usually the only ones published, are of little value in designing new plants. Relative costs cannot be given for the several types, but approximate

cost figures for Riensch-Wurl screens in Germany, as given to the writer by Mr. Wilhelm Wurl at the factory at Weissensee, are stated in Table 13. They show that the price increases with the size and capacity, and that the cost per million gallons capacity decreases.

With the drum or shovel-vane screen, it is possible to do away with the cost of a grit chamber, as these screens can handle this material as well as the screenings. The wear of combined sewage with its contained grit on a fine-mesh screen would make the removal of grit before screening desirable where these conditions obtain.

TABLE 13.—COST OF RIENSCH-WURL SCREENS.

Diameter.	Normal capacity, in cubic feet per second.	Price.	
		Per screen.	Per million gallons per day.
4 ft. 3 in.	0.6	$720	$775
5 ft. 0 in.	0.9	840	601
6 ft. 6 in.	.....	1 200	.....
9 ft. 10 in.	.....	2 400	.....
14 ft. 9 in.	14	3 600	166
16 ft. 6 in.	25	4 800	124
19 ft. 9 in.	35	6 000	110
23 ft. 0 in.	70	8 400	77.40
26 ft. 4 in.	160	12 000	48.40

Cost of Operation.—The costs of operation depend primarily on the power consumed, supplies, and attendance; but they also depend on depreciation, repairs, and renewals. The materials used should be durable, and subject to the least possible corrosion. The Windschild drum screen involves the expense of compressed air, but this objection may be counterbalanced by the decreased moisture in the screenings. The Riensch-Wurl and shovel-vane screens involve the periodical renewal of brushes. The latter type is not simple in design, and for this reason might cost more for repairs, but is believed to be well made. The Hamburg screen is subject to the wear on many moving parts. The Weand screen has shown indications of deterioration from corrosion and wear. Experience with these screens has been too brief to assign to each a probable period of life. One can only form an opinion in each case from its design and the conditions under which it is to operate.

FINE-SCREENING VERSUS SEDIMENTATION.

It has been shown by the data already presented that in the matter of efficiency the best fine-screening will compare well with such tank

treatment as may usually be looked for. That is, from 30 to 50% of the suspended solids may be removed by fine screens, as compared with 50 to 65% by sedimentation.

Emscher tanks have a particular advantage in producing a sludge which is inoffensive, even if stored for a considerable period, and may be used for filling in waste land or, possibly, as a filler for fertilizer.*

Tank treatment uses up little head and is subject to little deterioration, such as is inevitable with machinery of any kind, and although the cost of attendance may be a little greater, such attendance, except for general supervision, may be of a somewhat lower grade than is necessary when motors, screens, conveyors, and possibly boilers, engines, or air compressors have to be looked after.

Monti, after the Berlin experiments already referred to, concluded: "that although this fine screening removes all of the offensive-looking matter, yet the liquid remains quite turbid, and that much better results can be obtained by a few hours' sedimentation in large tanks."†

This is confirmed by G. M. Wisner, M. Am. Soc. C. E., who concludes, from experiments at Chicago,‡ that "there is little or no improvement in the stability of the local sewage due to screening through a device with 40 meshes to the inch."

On the other hand, tank treatment requires considerable area, and this is sometimes difficult to secure at moderate cost. It would be a great expense, for instance, in some parts of New York City. Then, with plain sedimentation, a large volume of watery sludge, which becomes very offensive if stored, must be disposed of, and this means an additional area for drying, with, perhaps, additional cost for artificial drying or for transportation to sea while in a crude condition. Emscher tanks require less land than other forms, but are more costly to construct, their great advantage being the favorable quality of the sludge.

The following extracts from a report by George W. Fuller, M. Am. Soc. C. E., to the Metropolitan Sewerage Commission of New York,§ are of especial interest, as coming from one who can speak with authority on these matters. He says:

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* "Fresh and Decomposed Sludge," L. C. Frank, *Engineering Record*, Vol. 68, p. 331.

† E. Kuichling, *Proceedings*, Thirty-second Annual Meeting, New Jersey Sanitary Assoc., 1908.

‡ "Sewage Disposal," by George W. Fuller, M. Am. Soc. C. E., p. 381.

§ Report of April 30th, 1914, pp. 209-210.

"Fine screens afford the cheapest way of removing visible objects of sewage origin from the waters receiving sewage where such screening treatment alone is sufficient for obtaining satisfactory results. Under conditions where the limit is at times reached in the amount of clarified sewage which a watercourse will oxidize satisfactorily, settling tanks as a general rule are cheaper to install than screens, because for a given cost they will remove a greater quantity of organic matter.

* * * * * * * *

"In my opinion, screens are preferable to settling tanks only where it is desirable or necessary to remove only relatively large sewage matters in suspension. Where settling solids would form deposits in the watercourses if screening alone were adopted, to install settling tanks will prove wiser than to install fine screens."

Screening plants, however, are very compact and, with grit chambers, need never occupy more than a fraction of a city block. The process is cleanly and largely automatic, requiring very little attendance. The product is much more readily handled, as it contains less moisture, as a rule, than sludge—from 75 to 85% instead of 90 per cent. It is true that some screenings may contain as much moisture as well-digested Emscher sludge, or perhaps more, but it is not, as in the latter case, in a liquid condition, but may be transported in ordinary wagons or cars after a brief period of draining.

If a recovery of grease or fertilizing value is considered, the dried screenings will be found, as a rule, to contain much more organic matter than the dried sludge, and, therefore, pound for pound, to yield a greater return. Unless the plant is of considerable size, the screenings can best be drained and then burned under boilers or in a special incinerator, plans for which, it is understood, are now under consideration by a suburb of New York City.

To sum up, although screens may remove less material than tanks, the difference is not very great in the best types of each, and still less when one considers the organic constituents, the presence of which is most objectionable in an effluent. The land required is much less than that required for tanks, and the cost of installation is probably less, although this depends on local conditions. As to gross annual charges, no general comparison is possible, but it is believed that screens will often be found quite as economical as tanks if the fixed charges are included and the total dried organic matter removed is used as a basis of estimate, in spite of the fact

that there is greater deterioration and that higher rates of wages may be required with screening.

The head lost in either case need not be great, except in the original design of the Weand screen, or where, as with the Carshalton screen, it is used to develop power for operation.

Conclusion.

The points which the writer has attempted to bring out may be summarized as follows:

The use of fine screens for sewage deserves more serious consideration than it has received heretofore in America. Although tank treatment will continue to be the standard preliminary process, under most conditions, from economic considerations, there are now several screens which will remove approximately as much of the organic suspended matter from sewage while occupying much less space and delivering a product that is both richer in manurial ingredients and more readily handled than tank sludge.

For the foregoing reasons, it is the writer's opinion that fine-screening will be adopted in the future by many towns situated on bodies of water which are capable of assimilating the effluent; possibly, also, as a preliminary process to tank treatment, filtration, and disinfection. On the other hand, the relatively high cost of attendance and the probable lack of a market for the screenings, when compared with conditions abroad, will probably serve to prevent the marked increase in their use in the United States which has been experienced in Germany.

In view of a reasonable development of fine-screening in America, more fundamental data as to efficiencies and costs are greatly to be desired. These are rarely available in their best form, even in Germany.

The volume of sewage received at the plant, or the population served, should be recorded, and also either (*a*) the suspended solids and total organic matter, or (*b*) the parts per million of average suspended solids in the crude sewage, the weight of the dried grit and screenings, and the percentage of each which is organic.

a.—The suspended solids and total organic matter in the crude sewage and after each operation should be recorded for the different times of the day and week, and then, if weighted according to the

flow when sampled, it will furnish the quantity of these essential materials that have been removed by the screens and such preliminary devices as coarse screens and grit chambers.

b.—A record should be kept of the parts per million of average suspended solids in the crude sewage, the weight of the dried grit and screenings, and the percentage of each which is organic.

Data of this kind from different sources would then be comparable and of much value in planning new works. The costs, too, not only of the screens, but of foundations, auxiliary machinery, and housing, with those for supplies and attendance, should be matters of record. So much stress laid on these things would perhaps be unnecessary, but for the fact that such essential data are, with few exceptions, not to be had.

Finally, the adoption of screens should be considered:

1. Where the principal requirement is the removal of the larger solid matters;
2. Where land values are high;
3. Where the cost of excavation for tanks is high;
4. Where the cost of sludge disposal is high, or where it is likely to cause objectionable odors; and
5. Where the recovery of grease and fertilizer is an important factor.

With openings not more than 0.10 in. (2½ mm.) in size, fine-screening should remove at least 30% of the suspended solids and 20% of the suspended organic solids from ordinary domestic sewage, or 0.1 cu. yd. of screenings, containing 75% of water, per 1 000 population daily.

The writer expresses his sincere desire that the many members of this Society who have studied the subject of fine-screening, will present such information and as many suggestions relative to the matter as possible.

Acknowledgment is hereby made for information furnished and other courtesies shown to the writer, rendering the preparation of this paper possible, by the following gentlemen and firms: Stadtbaurat Fleck, Dresden; Stadtbaurat Franze, Frankfort; Baurat Frensch, Wiesbaden; Dipl.-Ing. Keitel, Hamburg; Stadtbauinspektor Petersen, Cologne; Stadtbaumeister Schaefer, Frankfort; Betriebsinspektor

Scheitzow, Dresden; Dipl.-Ing. Friedrich Schlüssel, Osnabrück; Baudirektor Sperber, Hamburg; Abteilsbaumeister Schürman, Trier; Oberbaurat Steuernagel, Cologne; Stadtbaurat Strohl, Strassburg; Dipl.-Ing. Sturmfels, Elberfeld; Magistratsbaurat Uhlfelder, Frankfort; Magistratsbaurat in Gleiwitz; Baurat Willenz, Mainz; J. S. Fries Sohn, Frankfort; Geigersche Fabrik, Karlsruhe; Maschinenfabrik Buckau Actiengesellschaft, Magdeburg; G. Windschild, Dresden-Cossebaude; Wilhelm Wurl, Berlin-Wiessensee; Städtischer-Ing. Vidrighin, Temesvar, Hungary; Joseph Garfield, Sewage Works Engineer, Bradford; G. W. Humphrey, Chief Engineer, County Council, London; Mr. Beal, Superintendent, Southern Outfall Works, Crossness; Mr. Edw. S. Allen, Providence, R. I.; Mr. C. R. Felton, former City Engineer, Brockton, Mass.; X. H. Goodnough, M. Am. Soc. C. E., Chief Engineer, State Board of Health, Boston, Mass.; Mr. James Hayes, Jr., City Engineer, Brockton, Mass.; H. C. McRae, Assoc. M. Am. Soc. C. E., Division Engineer, Sewerage Commission, Baltimore, Md.; Mr. W. L. D'Olier, Philadelphia, Pa.; E. B. Ulrich, Assoc. M. Am. Soc. C. E., City Engineer, Reading, Pa., and Mr. O. M. Weand, Reading, Pa.

APPENDIX

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PAPERS AND DISCUSSIONS

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STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

BY MESSRS. H. T. CORY AND ALLEN HAZEN.†

Mr. Cory.

H. T. CORY, M. AM. SOC. C. E. (by letter).—Occasionally in life, one "comes to scoff and remains to pray." That, metaphorically speaking, is the writer's experience with Mr. Hazen's paper. In general, water storage required depends primarily on the run-off (including quantity and distribution throughout the year), and its estimation is a very dangerous and complex proceeding. The writer has seen such disastrous results from attempts to apply run-off data and run-off relationships from precipitation obtained in one section of the country to a locality or stream in another section, often near-by, that Mr. Hazen's paper was taken up in a spirit of impatience. So many tragic wrecks are strewn throughout the Far West from this sort of thing, that it is difficult to be tolerant of the elaborate methods constantly being suggested by ingenious mathematically inclined engineers for "deducing" discharges of ungauged streams. The instances cited by Mr. Taylor‡ are very pertinent, and the writer knows that the particular cases he mentions are presented fairly in his discussion.

It was largely a matter of luck, therefore, that the writer was led to glance over Mr. Hazen's paper in sufficient detail, before throwing it aside, to discover that it was not another and probably the most serious proposal yet made to apply mathematical methods to insufficient data in an endeavor to use observed run-off data covering a short period

* Continued from May, 1914, *Proceedings*.
† Author's closure.
‡ *Proceedings*, Am. Soc. C. E., January, 1914.

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of time, with a view to "expanding" them into records of sufficient length apparently to justify the design of hydro-electric engineering works to the third decimal place.

The paper, as a matter of fact, is of a totally different character. It was read with very great interest and profit—also with very considerable difficulty, in spite of the fact that the writer, as a student, was given the method of least squares more fully than is usual in American technical institutions, and, in addition, taught it and its applications in engineering work to his students for several years while in charge of engineering departments in two of the larger American universities. It is probable that most members of the Society found it still more difficult to follow the argument.

These two things are probably responsible for the little comment really pertinent to the paper itself which has been brought out. This small amount of discussion is greatly to be regretted, because the contribution, as a matter of fact, is most noteworthy, in several ways. The method of examining hydrographic data itself is of striking importance and interest, as well as the application of graphical methods to the theory of probability, or method of least squares, as it is more usually called.

The title and the context show that the author considers only storage for municipal supplies, and limits his conclusions to regions where hydrographic conditions are similar to those obtaining in the North Atlantic States. The method of analysis, however, is a general one, and the writer will consider its application to general hydraulic work in the Far West, where irrigation and power development overshadow municipal uses. From such a standpoint, some statements and methods of treatment, although quite justified in the paper as presented, require some modifications. The discussion which follows has to do very largely with such features, and, as a whole, is from that standpoint exclusively.

It is unfortunate that the first paragraphs of the paper were presented in just the form they were, because, at the very outset, they give the reader—at least they gave the writer—an erroneous conception of its scope.

"There is undoubtedly a definite relation between the storage provided in an impounding reservoir on any stream and the quantity of water which can be supplied continuously by it. The relation, however, is a complex one, and our knowledge of its character is limited. The following study is made to see how far it may be possible to separate this complex relation into parts, some of them being of such a nature that they may be studied separately with definite results, and afterward to treat all the remaining variations on the basis of probabilities, using all data from a number of different streams; and to study them in comparison with the normal law of error.

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"Among the elements that can be studied separately are the following:

"1. The size of the catchment area.

"2. The mean annual run-off per square mile.

"3. The portion of water area and the loss by evaporation from it. This relation is a complex one, and data for determining it are less adequate than could be desired. Nevertheless, some approximations can be reached.

"4. The natural storage in lakes, or in deposits of sand and gravel and other pervious materials. Only approximate results for natural storage can be reached, but as these are found to have a great influence on the required storage, especially at relatively low rates of draft, they must be considered.

"5. Regularity in flow. Some streams have comparatively regular flows; in others the variation is much greater. This difference in regularity of flow can be taken into account by finding a coefficient determined from the record of each stream, and bringing this into the statement in such a way that variations from the normal are stated in terms of the 'standard variation'. In this way, records of streams having more regular flows and those having less regular flows may be compared with reference to other matters.

"For the present, all remaining elements of variations of flow of every description will be thrown into one group and studied in connection with the normal law of error."

As a matter of fact, not "all remaining elements of variations of flow of every description" are studied in connection with the normal law of error, but, instead, all the essential elements of flow. As stated under the head, "Data Used," on page 1946:*

"The first step is to reduce existing data to the land-area basis, and make a tabular statement showing the average flow per square mile for each catchment area * * *."

That is to say, start with the actually observed run-off reduced to a uniform basis for comparative purposes, which, in itself, is involved and is the result of all five elements specified in the second paragraph, and a great many more. Indeed, with the actually observed run-off once known for any considerable period, the size of the catchment area is ordinarily a matter of indifference; the mean annual run-off per square mile, or otherwise stated, is one summation of many which can be utilized; and the regularity of the flow can at once be observed, or computed in terms of standard variation, or otherwise. The proportion of the drainage area which is water surface, the loss by evaporation from it, and the natural storage in lakes or deposits of sand and gravel, are important, but only primarily as affecting corrections of observed run-off data from several streams, in order to put them on a comparative basis.

The essential thing, then, is that the entire paper (and the method of analysis which it presents) concerns the study of run-off data reduced to a common basis—regardless of how such data may have

* *Proceedings*, Am. Soc. C. E., November, 1918.

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been collected—with the view of estimating what may be expected from dryer years or years when excessively low flow will occur. Although it is undoubtedly true that the same method could be used for examining precipitation data, and, on page 1976*, the author points out that dry years may be classified, with reference to the quantity of rainfall, the quantity of run-off, or the quantities of storage required to maintain certain drafts, nevertheless, as far as the author's presentation as a whole is concerned, actual adjusted run-off is the only thing taken into account and analyzed.

It is thus seen that the third paragraph of the paper as such is misleading, and may give quite a wrong conception of its subject-matter and add considerably to the difficulty of following the line of thought.

As the author states, the first requisite for a successful study of probabilities in run-off and in storage capacities from run-off data, is a sufficient number of facts. To obtain these, the method actually used is to combine the records of several streams reduced to a comparable common basis, thus forming a single series covering a great number of years. The objection to such a method is obvious, but, as a matter of fact, not so great as would seem at first thought. In the light of present knowledge, the combination of records of streams from distant areas having widely different conditions is not justifiable, and a series made up from a number of streams in a relatively small area reflects no changes in climatic conditions which might take place in a long term of years. Mr. Marsh† points out that the average daily run-off from the Croton water-shed in the 18 years, 1869-86, was 346 000 000 gal., and in the 18 years, 1887-1904, it was 449 000 000 gal. This difference is 30% of the smaller, and 23% of the larger, figure. As a matter of fact, however, in the treatment proposed in the paper, the safe draft and the storage unit are expressed in terms of the coefficient of variation, which latter does not vary materially for the two periods. If the draft is taken in this way, the variation in the mean annual run-off does not enter into the problem in so marked a manner. The actual draft in any absolute quantity units obviously does, however, and, in applying the results of the analyses explained by Mr. Hazen, one must be careful to realize that if the figure taken for the mean annual flow is not the correct one, the quantity figures will be also in error, the extent of the error when above the critical storage point being, according to the author, about 80% of that in the mean annual run-off (page 2015*).

The limitation of the method in this direction, however, does not take away its value for analyzing data of the character under consideration. Obviously, no method of examining data is conceivable which would promise quantity results any more dependable than the figures

* *Proceedings*, Am. Soc. C. E., November, 1913.

† *Proceedings*, Am. Soc. C. E., February, 1914.

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for mean annual run-off from water-sheds. Serious errors in mean annual discharge are much less, and can be more easily avoided in a dependable manner, than almost any other kind of errors in hydrographic records. It is undoubtedly true that the relation between the run-off in any one year and the rainfall of that year is impossible to establish within reasonable limits, because of the tremendously complex character of such relationship. The relation between mean annual rainfall over two long periods of time and the mean annual run-off for the same long periods of time, in the case of any given water-shed, however, is a very different matter. The writer does not know just what the facts in this particular are in Eastern, Northeastern, and Southeastern streams, but, in California, where wet and dry cycles run from 8 to 10 years in length, there is no difficulty in judging very clearly from long precipitation records whether or not such low-year run-off and high-year run-off cycles exist. It follows, therefore, that the author's conclusion that his method of studying such an artificial series seems to be the best means now available for finding approximately some of the relations between flows and storage, is absolutely justifiable.

On page 1946*, it is stated under the heading, "Land Area Only as Basis", that the "published figures are revised by dividing the run-off per square mile from the total area, by one less the proportion of water area." This may be an empirical rule found to be correct for the territory which Mr. Hazen was considering, but otherwise the reason for so doing is not clear. Mr. Marsh's comments on taking equivalent land area rather than gross area seem to be pertinent. On page 1950*, it is said that storage is most conveniently stated in days' supply of a maintainable yield. This is evidently true as regards storage for municipal purposes, but it is not true for irrigation and power storage.

It is probable that a great many members began to "read rapidly" —and not understandingly—at the heading, "Probability Paper." The author states that this is:

"* * * paper ruled with lines spaced in accordance with a probability curve, or, as it is otherwise called, the normal law of error. The spacing of the lines for this paper was computed from figures taken from probability curve tables, and arranged so that the line which represents the summation of the probability curve, when plotted on it, is straight."

This is a very short and interesting, but not satisfactory, description. As far as the writer is aware, this is the first time, in engineering literature at least, where probability paper has ever been used, its use suggested, or its preparation even touched on. The short description thus given is, to say the least, not comprehensive, and, indeed, some "arrangement" of the figures taken from probability curve tables must be made in order to obtain the probability paper as given. Just what that arrangement is, the writer has not had time to figure out, and he can well understand that most engineers are

*Proceedings, Am. Soc. C. E., November, 1913.

Mr. Cory. not willing to use a new mathematical or graphical tool without at least understanding how it is prepared. The equation of a straight line when plotted on such paper would be interesting. It is obvious that in going into an entirely new field, as the author has in this paper, great conciseness is necessary in order to keep the contribution within reasonable length, but it is hoped that the discussion will bring out a little more information and detail regarding this new probability paper.

The definition of dry years is an excellent idea, and it is well to emphasize the fact that any given percentage dry year must be considered as a type, and not any one actual year.

On page 1978*, the author says:

"The importance of making the correction for daily results, when monthly records are used for the basis of calculation, will be realized when it is stated that in most cases this correction amounts to more than the allowance for evaporation."

Although this is practically true of the Eastern reservoirs discussed by the author, it is very far from being true in irrigation practice west of the Rocky Mountains. Indeed, in California, the evaporation loss from municipal reservoirs often amounts to more than 25%—being so great sometimes that the hardness of the water at the end of dry cycles is more than doubled. The same applies to the second paragraph on page 2007*, in which it is stated that the correction for the quantity of storage in the Croton water-shed, estimated by Mr. Freeman some years ago, deduced by the method of analysis given in the paper, is 30%, "* * * an addition so large that it overshadows all allowances for evaporation, ground-water storage, and other secondary conditions * * *".

For Eastern conditions, where the annual precipitation directly on water areas is approximately equal to the annual evaporation therefrom, the author's methods of handling evaporation, as given on page 2032*, is doubtless satisfactory. However, where the evaporation is such a very important factor as it is in the Far West, it is absolutely necessary, in order to avoid very serious mistakes, to eliminate this item on both sides of all equations, and regard it as a part of the total draft, that is, the maintainable draft should be considered as the gross draft, and the net draft should be determined from it and the evaporation, using very short intervals of time and actual water areas for each of the respective time intervals. This is particularly true in the case of irrigation reservoirs, where the draft is not regular throughout the entire year but is concentrated throughout half the year or less, with a maximum daily draft frequently twice that of the average even for the short irrigation season.

As this is the case, in order to make the method general, it seems to the writer that regarding evaporation as one part of the total draft,

* *Proceedings*, Am. Soc. C. E., November, 1913.

just as the net draft is the other part, instead of using the method which the author suggests, is absolutely essential. Mr. Cory.

On page 2000*, the paragraph: "It is probable that 2 or more of the 5 years would follow consecutively in one period of several years of low average flow", might be misleading, because the method takes into account this very probability of a succession of dry years.

The paper would be a little clearer and more easily read were parenthetical descriptions of the kind of paper in the various figures indicated, arithmetic-arithmetic, arithmetic-logarithmetic, arithmetic-probability, and logarithmic-logarithmic papers being used without stating which is which.

On page 1996*, the author states that in the Eastern streams investigated, the annual flow will fall to 55% or less of the normal, once in 100 years. On the other hand, in Western streams it frequently falls to zero for 2 or 3 consecutive years in the case of some important streams. Indeed, the coefficient of variation of the Sweetwater River at the Sweetwater Dam is considerably greater than unity, being 1.38.

On page 1997*, the author states:

"It is obvious that the storage required to balance annual fluctuations in flow will increase with the coefficient of variation for the mean annual flows. It is not unlikely that the coefficient of variation may be used as a basis for measuring the storage in such a way that the results will be general, and will apply equally to all the streams within the range covered by this study."

With the view of ascertaining what results would be obtained by applying the method to some of the most variable of the Western streams, the writer examined the run-off data shown in Fig. 38—probably the most variable of any streams in the United States—and that of another series totaling 142 years from another locality (details not given) to see whether the coefficient of variation might be used as a basis for measuring storage. The results were startling, and are given graphically in Fig. 39. Unless it is a case of mere coincidence, which seems improbable, it appears that, not only the method, but this latter curve, can be appled to all streams within the range covered by the writer's study, as well as to all those within that covered by the author.

This is most significant. It indicates that the author has proposed a method by which is obtained a wonderfully effective tool for the use of hydraulic engineers—the figure showing storage units for various values of k for 80, 90, 95, 98, and 99% dry years, and is Fig. 30 extended as low as $k = 0.1$, through the examinations which the writer has just mentioned as having made. In this curve, the storage units equal the standard variation or the coefficient of variation multiplied by the mean flow, and k is the coefficient in the expression:

"Draft $= (1 - k \times$ Coef. of variation$)$ = Portion of mean flow used."

* *Proceedings*, Am. Soc. C. E., November, 1913.

Mr. Cory.

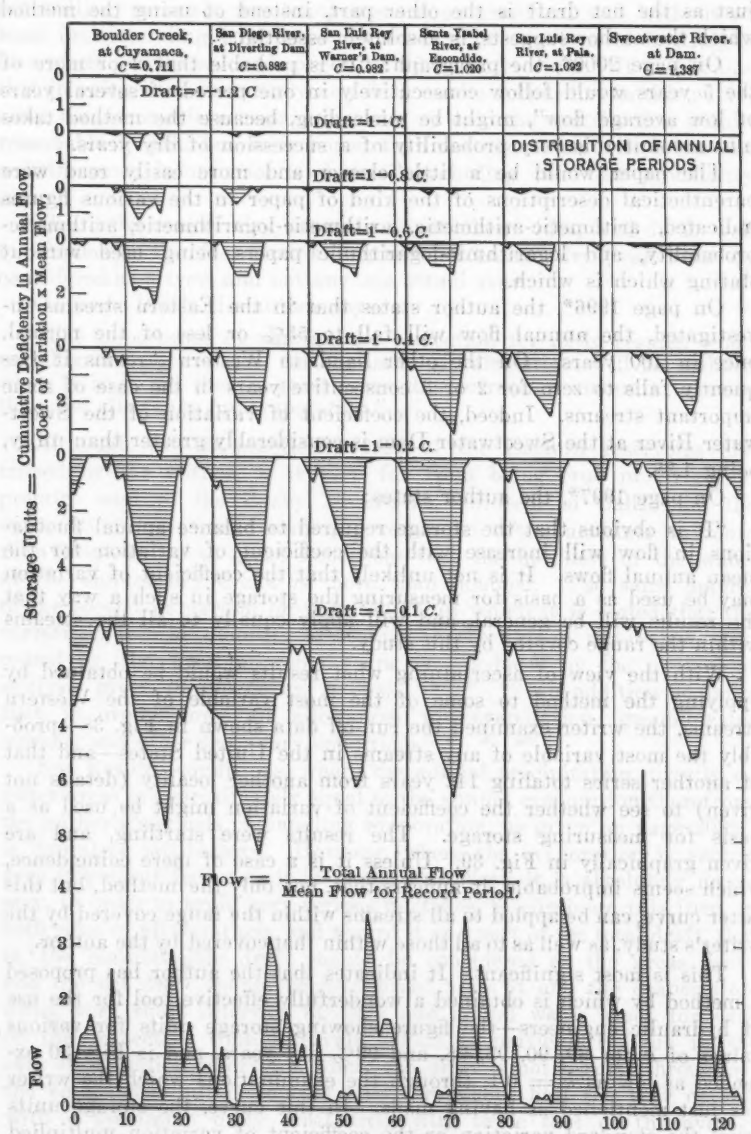

FIG. 38.

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It seems premature to suggest that this curve will hold true for streams or sets of streams in general localities, widely separated, and having excessive differences in run-off characteristics. At the same time, the curves do represent very closely the results obtained by the author in the streams which he examined in the New England and neighboring States, and equally well the data from some streams in the very southeastern corner of California and from some in its central portion. In the latter region, the run-off characteristics are quite as different from those in the southern end of California as they are from those selected by the author from New England territory.

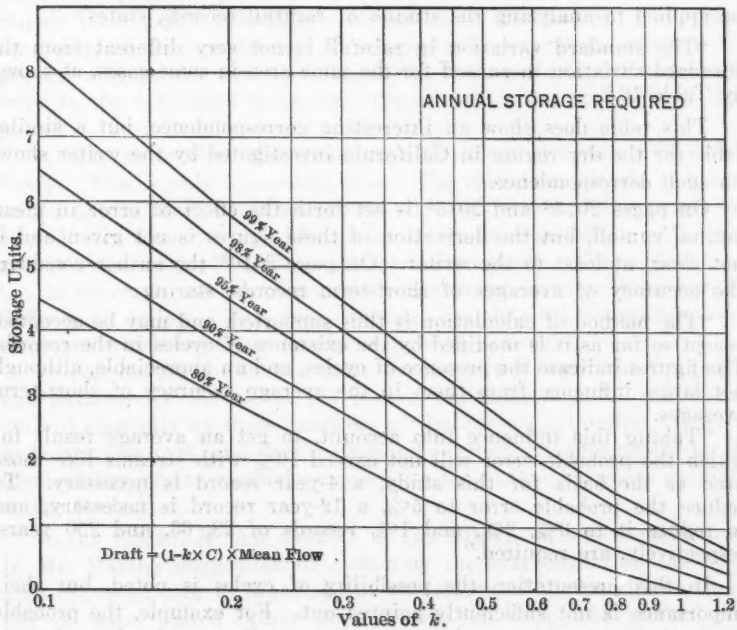

FIG. 39.

Should further investigation show that these curves are really general in their application and that their coincidence in the case of these exceedingly different sets of streams is not a mere matter of chance, the ability to determine very quickly the available draft from any stream with a given quantity of storage, or *vice versa*, by a half day's study of its yearly run-off data, will prove of almost incalculable value to the Profession. It is for this reason and for the desirability of determining quickly the limitations of the method and the propriety of its general application, that the small amount

Mr. Cory. of discussion which the paper has brought out is especially to be regretted.

It is important to remember that to the quantity of storage, as given by the curves in Fig. 39, must be added the actual monthly storage (corrected for daily storage) required to tide over dry seasons in the year, the estimation of which is considered in that part of the paper preceding page 1996*. Such monthly storage appears to be about 245 days for the complete development of the Eastern streams investigated by the author, and about 320 days in the California streams investigated by the writer.

On page 2009*, the author, having explained that the method can be applied to analyzing the studies of rainfall records, states:

"The standard variation in rainfall is not very different from the standard variation in run-off for the same area in some cases, as shown by Table 16."

This table does show an interesting correspondence, but a similar table for the dry region in California investigated by the writer shows no such correspondence.

On pages 2014* and 2015* is set forth the effect of error in mean annual run-off, but the derivation of these figures is not given and is not clear, at least to the writer. On page 2016* the author considers the accuracy of averages of short-term records, stating:

"The method of calculation is thus supported, and may be accepted, except so far as it is modified by the existence of cycles in the records. The figures indicate the presence of cycles, and an appreciable, although not large, influence from them in the average accuracy of short-term averages.

"Taking this influence into account, to get an average result for which the probable error will not exceed 10% with streams like those used as the basis for this study, a 4-year record is necessary. To reduce the probable error to 5%, a 12-year record is necessary, and to reduce it to 3%, 2%, and 1%, records of 28, 63, and 250 years, respectively, are required."

In this presentation the possibility of cycles is noted, but their importance is not sufficiently pointed out. For example, the probable error of the 18-year record, 1887-1904, according to the foregoing, is less than 5%, yet, according to Mr. Marsh's statement in his discussion, this average on the Croton water-shed was 23% greater than the 18-year period, 1867-86. Of course, the variation from the total mean is much smaller, but the safe dependable yield throughout the earlier period of 18 years is obviously the thing, and practically the only thing, of interest to hydraulic engineers in estimating dependable yield, particularly for municipal purposes. With regard to this point, it seems to the writer, as was expressed early in this discussion, that there was no use in hoping to obtain results by analyses of run-off records any more reliable than the figure for the mean annual run-off

* *Proceedings*, Am. Soc. C. E., November, 1913.

itself, and that the possibility of such dry cycles must be examined by a critical analysis of the precipitation records over the same general region, which ordinarily extend back very many times as long as run-off observations. Mr. Cory.

On page 2020*, in the equation for the probable error in the calculated maintainable yield of an Eastern stream above the critical point,

$$\text{Probable error} = \frac{0.55 \times \text{coefficient of variation}}{\sqrt{\text{number of years in record}}},$$

the reason for the use of the figure 0.55 is not clear to the writer. Also, the author's discussion: "Actual Probability of Shortage in Supply", on pages 2038-2040*, is not clear, particularly the reason for using 0.8 as a multiplier of the probable percentage error in the general average for the whole series, in order to obtain the probable error in the calculated mean flow, the storage being above the critical point.

It may seem that the writer has criticized the paper somewhat freely. This is only apparently true. The criticisms are all of a very minor nature, and would not be at all worth while if the paper was of ordinary importance. The presentation is remarkably well done, and is a fine example of clearness and conciseness. The subject matter, however, is not simple, and the argument is more than usually difficult to follow. For these reasons, the detailed suggestions and comment which have been made seem to be desirable.

In conclusion, the writer again expresses the hope that the paper, and particularly the use of the revised curves in Fig. 39, will be examined critically by those members of the Society interested chiefly in hydrographic work, and that their results and experiences regarding the applicability of the method proposed by the author be given promptly to the Society.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—There is much force in Mr. Marsh's suggestion of reckoning the evaporation on the water area corresponding to a storage of two-thirds of the maximum quantity of water stored rather than on the full area. The method suggested by Mr. Cory of considering the evaporation as part of the draft is ingenious, and seems to offer substantial advantages for use in dry climates and with long periods of depletion. For Eastern conditions, where, as an annual average, the evaporation is ordinarily less than the rainfall, but considerably exceeds it during a certain period in summer and fall, the method suggested in the paper seems to be better adapted. Mr. Hazen.

There is also the interesting case, occasionally found (not discussed either in the paper or by Mr. Cory), where the water drawn from existing lakes or reservoirs is measured after evaporation has taken place; and, where this is the case, obviously, no further allowance

* *Proceedings*, Am. Soc. C. E., November, 1913.

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for evaporation is required in computing the supply that can be utilized from such a source. When, however, it is desired to compare the results with records of flow from streams which have not yet been impounded, allowances for evaporation are necessary in order to estimate the probable natural flow of the stream. This procedure was followed in preparing the Croton, Sudbury, and Wachusett data used in the paper.

After all, the different methods of allowing for evaporation are only different ways of doing the same thing, and, with the same fundamental assumptions, no appreciable difference in results should grow out of any reasonable variation in the method of making the correction.

It is interesting to note that further study of evaporation indicates that, in most cases, early estimates were too high. In other words, the errors in the experimental procedure have usually been in the direction of increasing the apparent quantity of evaporation.

The question of whether the land area or the total area should be used as a basis for stating run-off is one which does not affect in any way the general method proposed by the writer. It makes no difference in the final result which of the two methods is used, so long as that method is kept clearly in mind and is properly applied to the end of the calculation. The land area was selected for use by the writer, for Eastern streams, following Mr. F. P. Stearns' classic discussion of this subject, and he still believes that for these streams this method is, on the whole, rather more convenient. In other cases, it has no advantages.

Mr. Marsh's statement in regard to the mean flow of the Croton for two successive 18-year periods is interesting. Obviously, such periods of low flows must be anticipated from time to time, but how often? The method outlined in the paper gives the basis for a definite answer to this question. The coefficient of variation in annual flows of the Croton River is 0.239. The probable error in any one 18-year period is:

$$\frac{0.239 \times 0.6745}{\sqrt{18}} = 3.8 \text{ per cent.}$$

In the 18-year period from 1869 to 1886, the mean flow was 12.5% less than the record mean for the whole period. Assuming the record mean to be the true mean, the actual error for this 18-year period was 3.3 times as great as the probable error for one 18-year period. From probability tables it appears that 97.4% of all the terms in a series which follows the normal law of error, differ from the mean by less than this amount. The remaining 2.6% of the terms are outside the limits, one-half being above the upper limit and one-half below the lower one. Of all consecutive 18-year series, 1.3%, therefore, would be expected to be less than the true mean by an amount at least as great as that which this Croton series did actually drop

below the record mean of the Croton River. In other words, in any 18-year series, there is 1 chance in 77 that the mean will fall below the true mean by as much or more than the actual reported drop in this case. In an indefinitely long period of years, one such 18-year dry period would be expected once in every 77 years, on an average. The Croton records cover 46 years, to the present time. Within this record are comprised 29 periods, each having 18 consecutive years. Each of these periods had 1 chance in 77 of being below the limit. There were, therefore, 29 chances out of 77 that some one of the 18-year periods would fall short of the record mean by an amount at least equal to the shortage that is reported in this case. The probabilities are almost 2 to 1 that such a shortage will not happen in any given 46-year period. The variation in this record, although it reaches 3.3 times the probable error for any one 18-year series, is one that would have to be expected in one out of every three 46-year periods, and its actual occurrence in this case is an impressive example of the possibility of error in short-term averages. Mr. Hazen.

It is possible, of course, that in this case there may be a long-term cycle variation in rainfall and run-off which may contribute in some measure to the low average, and it may also be mentioned that the greater part of the flow in the first 18-year period was measured as flood discharge over the old Croton Dam, and the greater part of the flow in the second 18-year period was measured in the two Croton Aqueducts on its way to the city; and it may be that, notwithstanding the great care that has been exercised in adjusting the old records, some small residual error may exist and be an element in the reported difference.

Answering Mr. Mills' question, the method proposed is an elastic one and allows the degree of provision against drought to be selected. By working to the 99% year, shortages such as those which Mr. Mills speaks of will be guarded against in 99 years out of 100, though, if the 95% year is selected, a shortage must be anticipated once in 20 years. Therefore, any degree of insurance that is desired can be selected, and the calculation made accordingly.

Mr. Taylor is entirely right in calling attention to the wide difference in run-off and storage conditions between the East and the West, and it is unfortunate that these differences have been so often overlooked, and that such investments as the one which he mentions have been made on inadequate data. On the other hand, as he points out, even in the same case, a properly designed development would have in it great possibilities of good, and would no doubt ultimately prove remunerative.

Mr. Tighe has recorded the origin of the mass-curve, and it may be well to point out that the methods now proposed do not in any way supersede it. On the contrary, the mass-curve method is used as

Mr. Hazen.

a basis for computing the storages on which all the tables and diagrams in the paper are based. Practically, however, the mass-curves were not drawn, because it was found that the results might be obtained more easily arithmetically.

Mr. Tighe points out an exception to the rule that, within certain limits, storage on a tributary is substantially as useful as on the main stream. The summer refill water which he mentions, is small in quantity in the dryest years; it is much more abundant in wetter years, when it is not needed. There is no doubt that a small storage on a remote site would have reduced value because of the loss of water in transit in a dry time. With abundant gravel, water lost by percolation may be more important in this case than water lost by evaporation, and, although the former may be expected for the most part to re-appear subsequently on the main stream, it may come at a later time, when the natural flow is otherwise sufficient, and when it is not needed. So far as this is the case, the stored water will be lost and the available supply decreased. The loss of water by percolation extending to and around the intake dam is not taken into account in the method of analysis proposed in the paper. If it were desirable to consider such loss, it would seem best to regard the water lost in this way as a draft, in the same way that Mr. Cory proposes to treat the loss by evaporation.

Mr. Goodrich has suggested a possible explanation of the fact that the curves representing storage and flow data are skew curves, and do not exactly follow the normal law of error. It is his thought that the curve may be composed of several independent curves superimposed on each other. This matter is an extremely important one. It seems to carry great possibilities of improved methods of analysis. The writer earnestly hopes that Mr. Goodrich will find time to develop this method and apply it, and he suggests that this subject would be a most appropriate one for a paper to be presented to the Society.

Mr. Cory's discussion is of fundamental importance. It greatly increases the value of the paper, because it extends the use of the methods to new fields and shows their convenient and useful application. The writer regards the method as more important than the results deduced by it and presented in the paper, and he is particularly interested, therefore, in showing that it is equally applicable in California and in New England, and to problems of municipal water supply, irrigation, and power.

In some cases, at least, the actual numerical figures obtained on a general basis from the records of Eastern streams having the smallest variations in their annual flows are found to be substantially equal to like figures obtained by applying the same methods to Southern California streams having the greatest variations in their annual flows. This coincidence is striking, and it cannot fail to add con-

fidence to the use of the method for all intermediate conditions, and these include most of the cases that occur. It was the intention to make the methods general in form, so that they would have wide application, but the discovery that both the method and the figures in some of the diagrams do apply to such widely different conditions as those of Southern California is most gratifying. Mr. Hazen.

Fig. 30 is extended far enough to include all the storage ordinarily used, or that will ordinarily be desired on such Eastern streams as serve as the basis of the study; but, in California, where water is relatively more valuable, and where cheap storage sites are found, the economical limit of storage is moved upward many times, as is also shown in Mr. Le Conte's discussion; and many of the larger water projects in successful use in California at the present time will be found to be beyond the limits represented by Fig. 30. The extension of the study to these higher relative storages, therefore, is desirable and necessary. Mr. Cory, using California data, has extended it in this way.

The writer has checked Mr. Cory's results by an independent study of some California data and by extending the Eastern results, and he is satisfied of their substantial accuracy. For the lower relative storages and rates of draft, the Eastern data presented in the paper are adequate to determine the required storage with a fair degree of certainty, and they are better suited to this use than Mr. Cory's California records. On the other hand, the California data for the higher rates of draft are better than the Eastern data, and are to be preferred. The combination diagram (Fig. 38) which Mr. Cory has prepared, therefore, is a safer one to use than could have been deduced from either set of data taken by itself. The fact that the two sets of data produce results which check at the point of junction, both in position and in direction, is most satisfactory.

It is not to be supposed that the storage ratios for the highest rates of draft can be determined with the same degree of accuracy that is reached with the lower ones. With a draft equal to $1 - 0.4C$*, which was the highest limit investigated with the Eastern data, there will be, on an average, a period of about 9 consecutive years in each 20 when there is no overflow. With the records covering 20 or 30 years, and in a few cases somewhat longer periods, this is about as far as the analysis can be carried without sacrifice in accuracy. With the greater draft of $1 - 0.2C$ and $1 - 0.1C$, considered by Mr. Cory, and necessary to include California data and conditions, the normal periods of depletion are greatly extended, until the normal condition of the reservoir becomes one of being only partly full. It is the exception, perhaps the rare exception, when the water reaches the overflow. This condition is actually reached in many California reservoirs

* Mean flow multiplied by 1 less 0.4 coefficient of variation.

Mr. Hazen.

at the present time. The greatest normal period of depletion in a 100-year period may rise to 40 or 60 years, or more. It is obvious that records of streams covering only 20 or 30 years are inadequate to establish with certainty the full storage required for these higher rates of draft. The writer, therefore, regards the figures in Mr. Cory's extended diagram (Fig. 38) as approximations, which are the best available at the present time, but are certain to be modified more or less when more extended data become available.

The question may also be considered whether, by mathematical methods and a better analysis of the probabilities of the situation, it may not be possible for some one to reach a better understanding of the matter than the writer has been able to do.

To give Mr. Cory's extended diagram (Fig. 38) full use, and to show its application, not only to California streams, but to many others having higher coefficients of variation than any that were discussed in the paper, the writer presents Figs. 40 and 41, as representing the storage required to maintain the supply in 95% and 98% dry years for California and other similar conditions. These diagrams are based on Mr. Cory's extended diagram (Fig. 38) with a curve for monthly and daily storage based on some studies by the writer of a few California streams. It is the writer's belief that this will be found to fit well enough to be used with advantage in estimating storage in a great variety of climates. It is especially applicable where the variation in annual flows is greater than that for Eastern streams, and where the seasonal fluctuation in flow is also greater. Figs. 40 and 41 also differ from Figs. 33 and 34 in extending much farther upward, to cover streams with greater variations in flow, and with the use of greater relative storages.

The most careful and conservative estimates of public water supplies for Eastern cities in the last decade have been based on assumptions substantially as severe as those corresponding to a 98% dry year. It is the writer's opinion that the 95% year will be more generally applicable, and, on the whole, is a better basis for rating works. If a basis were used corresponding to the 95% year, and it were wrongly assumed that it represented the dryest year to be expected, great damage might follow, but if the limit is recognized and kept always clearly in mind, the likelihood of harm will be largely eliminated. The use of the term, 95% dry year, is in itself a constant reminder that 1 year in 20 will probably be dryer than it, and therefore tends to safety. On the other hand, to limit the rating of works by using the 98% year as a base has the effect of reducing the available supply so much that all of it can be used on an average only once in 50 years, and this will seldom be justified.

The writer regrets Mr. Cory's difficulty in understanding portions of the paper. This may be due to the writer's effort to compress the

Mr. Hazen.

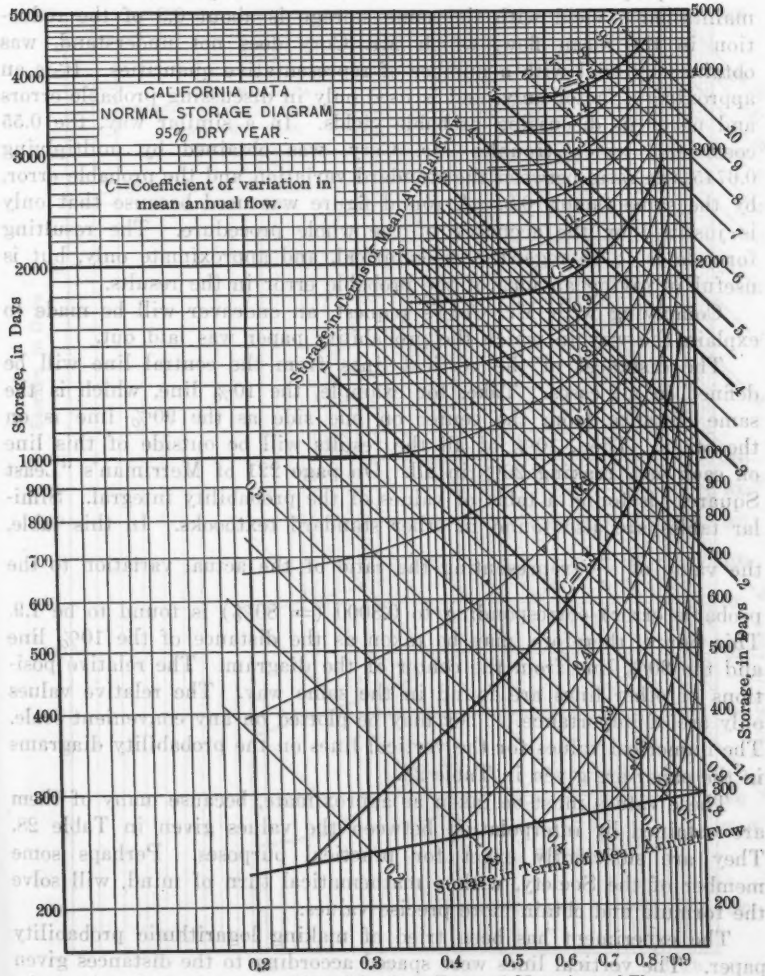

FIG. 40.

Mr. Hazen. material so as to keep the paper within reasonable limits of length, and to avoid making it a treatise on least squares and statistics, and covering ground better covered in textbooks on those subjects.

The proposition that above the critical storage the reduction of maintainable yield with the same storage is about 0.8 of the reduction in the mean flow, which Mr. Cory does not understand, was obtained by trial with a number of representative quantities. It is an approximate factor only, and is used only in discussing probable errors and never in discussing probable yields. In a similar way, the 0.55 coefficient, which troubles Mr. Cory, was obtained by multiplying 0.6745, the ratio between the standard variation and the probable error, by the same factor, 0.8. A round figure was used because that only is justified by the accuracy of the whole procedure. The resulting formula is to be regarded as empirical, and approximate only, but is useful in giving an idea of the probable error in the results.

Complying with Mr. Cory's request, an endeavor will be made to explain the way in which the probability paper was laid out.

The distances of the vertical lines from the central line will be defined in all cases. Take, for example, the 10% line, which is the same distance from the center on one side as the 90% line is on the other. Then, 10% of all the results will be outside of this line on each end, making 20% in all. On page 221 of Merriman's "Least Squares" is found a table of values of the probability integral. Similar tables are also found in other standard textbooks. In this table, the value of $\frac{x}{r}$ (representing the ratio of the actual variation to the probable error) corresponding to 0.8000 (= 80%) is found to be 1.9. This figure, therefore, may be taken as the distance of the 10% line and the 90% line from the center of the diagram. The relative positions of other lines are found in the same way. The relative values only are of importance. They may be plotted on any convenient scale. The numerical values for the vertical lines on the probability diagrams in the paper are given in Table 28.

These values must be taken as approximate, because many of them are obtained by interpolation between the values given in Table 28. They are sufficiently exact for practical purposes. Perhaps some member of the Society, with a mathematical turn of mind, will solve the formula and obtain more precise values.

The experiment has been tried of making logarithmic probability paper. The vertical lines were spaced according to the distances given in Table 28, and the horizontal lines were on a logarithmic scale. Numbers plot in a straight line on such paper when their logarithms form a true probability series. Logarithmic probability paper has been found helpful where the degree of skewness in the curve is very great. For the flow and storage data used in the paper, the

Mr. Hazen.

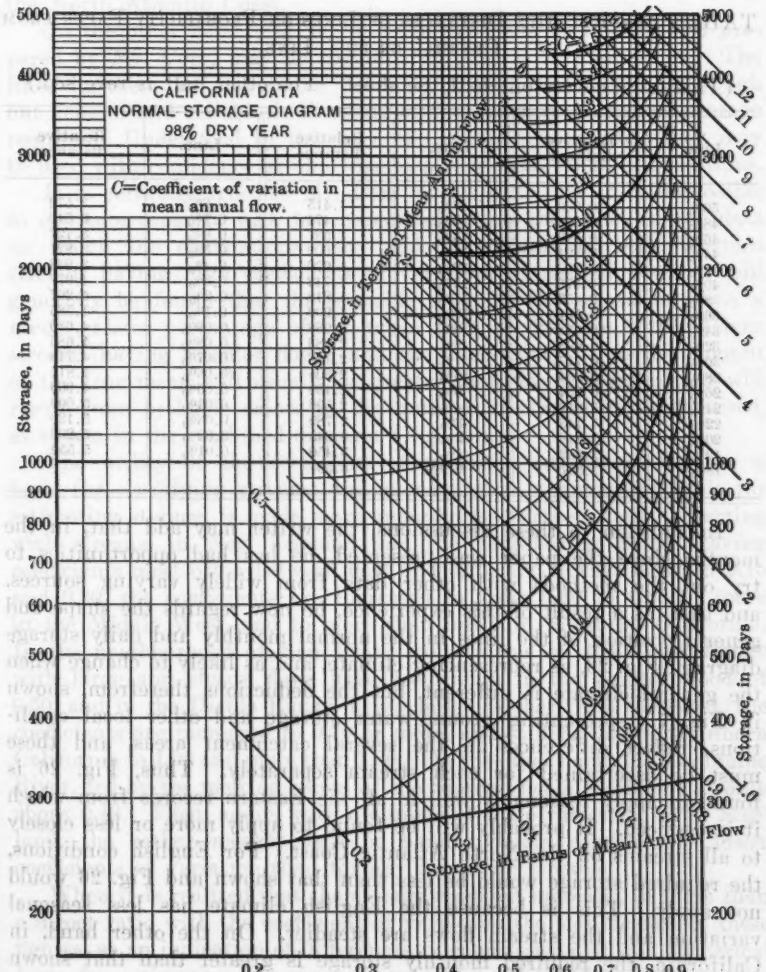

FIG. 41.

Mr. Hazen.

arithmetic probability paper has proved more satisfactory. For some other investigations the logarithmic probability paper has seemed to be better.

TABLE 28.—RELATIVE DISTANCES OF LINES ON PROBABILITY PAPER FROM CENTRAL OR 50% LINE.

(This table covers one-half the sheet. The other half is reversed.)

Line.	Relative distance.	Line.	Relative distance.	Line.	Relative distance.
50%	0.000	17%	1.415	0.8%	3.573
48%	0.074	16%	1.474	0.7%	3.646
46%	0.149	15%	1.537	0.6%	3.727
44%	0.224	14%	1.602	0.5%	3.821
42%	0.300	13%	1.670	0.4%	3.933
40%	0.376	12%	1.742	0.3%	4.077
38%	0.453	11%	1.818	0.2%	4.267
36%	0.531	10%	1.900	0.1%	4.585
34%	0.611	9%	1.988	0.09%	4.630
32%	0.693	8%	2.083	0.08%	4.685
30%	0.777	7%	2.188	0.07%	4.748
28%	0.864	6%	2.305	0.06%	4.817
26%	0.954	5%	2.439	0.05%	4.900
24%	1.047	4%	2.596	0.04%	5.000
22%	1.145	3%	2.789	0.03%	5.120
20%	1.248	2%	3.045	0.02%	5.290
19%	1.302	1%	3.450	0.01%	5.550
18%	1.357	0.9%	3.507		

In addition to these discussions, the writer may add that, in the months since the paper was presented, he has had opportunities to try out the methods with other data from widely varying sources, and that, as a result of this experience, he now regards the shape and general position of the lines in the normal monthly and daily storage diagram, Fig. 26, as representing climate, and as likely to change when the general climate is different, but the deductions therefrom, shown in Fig. 7, representing ground-water storage and other local conditions, reflect differences in the several catchment areas, and these must be ascertained for each stream separately. Thus, Fig. 26 is found to apply reasonably well to all the Eastern records from which it is deduced. It probably will be found to apply more or less closely to all streams on the North Atlantic Coast. For English conditions, the required storage would be less than that shown and Fig. 26 would not apply. This is because the English climate has less seasonal variation and the stream flows are steadier. On the other hand, in California, the required monthly storage is greater than that shown by Fig. 26. This figure, therefore, may be accepted only for the general region from which the data were obtained.

For other climatic conditions, the local data could be made to furnish, by the methods described on pages 1950 to 1977,* a diagram of normal daily and monthly storage for local use.

* *Proceedings*, Am. Soc. C. E., November, 1913.

Mr. Hazen.

As Fig. 26 serves as the base of Figs. 33 and 34, these diagrams should also be regarded as only strictly applicable to regions where the general climatic conditions do not differ widely from those on the North Atlantic Coast.

On the other hand, Fig. 30, and the extension of it (Fig. 38) prepared by Mr. Cory, may be regarded as general in application. The lines in them are not to be taken as precise, but rather as rough but general and reliable indications of the quantity of annual storage required. Understood in this way, the writer believes that they may be used with confidence as applying to a very wide range in conditions.

It is certainly true, as Mr. Cory points out, that it is impossible to estimate capacity with less percentage error than that in the data on which the calculations rest; and, in connection with Western streams having high coefficients of variation in annual flows, it will generally be found that the probable error of the record mean is a much greater percentage of the mean than is the case with Eastern streams having steadier flows; and the uncertainty as to the amount of the true mean will usually be greater than any difference that will result from probable errors in the number of storage units required, as shown in the combined diagram (Fig. 38).

The utility of the coefficient of variation in annual flow as a basis for classifying streams and as serving as a numerical basis for estimating storage, is made more apparent by further study. Starting with streams in New England and the neighboring States having coefficients most often between 0.20 and 0.25, and always between 0.15 and 0.30, the coefficients are increased to from 0.40 to 0.60 or more in the Middle West, and to more than unity in Southern California. In the neighborhood of San Francisco, streams on the Coast Range with large and steady flows have coefficients in the neighborhood of 0.50, and in the Diablo Range, a few miles farther from the coast, the coefficients are increased to 0.70 or more, and still farther in Southern California to 1.00 and more than 1.00. On the other hand, the Tuolumne River, fed by the melting snows of the Sierras, has a coefficient of only about 0.42. Then, again, with a climate of little variation, as in England, the rather meager data at hand indicate considerably lower coefficients of variation than for any American streams.

When it is realized that the storage required increases in more than a direct ratio with the coefficient of variation, the significance of these figures, as an indication of differences in conditions, and as a numerical basis for estimating the storage required in the different cases, may be realized.

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On the other hand, Fig. 30, and the extension of it (Fig. 35) prepared by Mr. Cory, may be regarded as general in application. The lines in them are not to be taken as precise, but rather as rough but general and reliable indications of the quantity of annual storage required. Understood in this way, the writer believes that they may be used with confidence as applying to a very wide range in conditions.

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PAPERS AND DISCUSSIONS

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GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

By Charles H. Paul, M. Am. Soc. C. E.

Mr. Paul.

Charles H. Paul, M. Am. Soc. C. E. (by letter).—The author has presented much interesting information, and there is no doubt that this paper, together with the discussions that it calls forth, will prove a valuable addition to engineering literature.

Foundation grouting is being carried on in connection with the construction of the Arrowrock Dam, and, although conditions there are not comparable with those at Estacada, a brief discussion of methods and results may be interesting in connection with the general consideration of this subject.

The Arrowrock Dam is a concrete masonry structure of the gravity type, 350 ft. high. The general level of the foundation in the canyon section is about 80 ft. below the river bed, and reaches a depth of 90 ft. at the deepest point. The foundation is a firm, hard granite. The rock is free from evidences of faulting, crushing, or shearing, but is characterized by rather frequent parallel joints giving it a sheeted structure which has a general north and south (across-canyon) trend, and a high dip up stream. On account of the unusual height of the dam, it was desired to take every precaution to prevent excessive seepage or the accumulation of upward pressure. Therefore, it was decided to grout the foundation and abutments along the up-stream face, although it was believed at the start that the character of the rock was such that the seams would take very little, if any, grout. In other words, this grouting was looked on from the beginning simply as a precautionary measure, and it was believed that the integrity of the structure would not be affected seriously by the success or failure of the grouting operations.

* Continued from May, 1914, *Proceedings*.

Mr. Paul.

Along the up-stream face, a cut-off channel, or keyway, with a minimum depth of 10 ft., was cut out of the bed-rock, and the grout holes were drilled from the bottom of this channel to a depth of 25 or 30 ft., thus making the bottom of the holes from 35 to 40 ft. below the general level of the foundation. Two rows of holes were drilled, 5 and 13 ft., respectively, inside the line of the up-stream face of the dam, and the longitudinal distance between the holes in each row was 10 ft., with the holes in the two rows staggered.

At the beginning of the work, these holes were drilled before the rock was covered with concrete, and were brought up through the concrete, by a 3-in. wrought-iron pipe, to the elevation from which they were grouted. This pipe was set in such a way that, when grouting, the mixture could flow freely into any seam that might exist between the concrete and the rock. It was realized at the time that the most approved method for work of this nature was to drill and grout a hole before another hole was drilled in that immediate vicinity, but that plan was not followed in this case for two reasons: First, the absence of large seams and the apparent tightness of the existing seams in the bed-rock appeared to make it unnecessary to carry out the work in this manner; second, bed-rock was not uncovered until late in the fall, and as the season for placing concrete was nearly over, and it was most desirable to have the bed-rock covered with concrete before winter set in, it was necessary to push the concreting without any loss of time. Later developments proved that in this case this method of grouting was justified, as in only a very few cases, when the leakage test was applied to the hole before grouting, was there any connection between the hole tested and adjacent holes. In the few cases of this kind that did occur, a satisfactory job was obtained by grouting the two or three connecting holes in as close succession as possible, and capping the other connecting holes while one was being grouted.

During the early part of the work Burley drills were used. Later, after the foundation had been covered and the grouting operations were carried on up the abutments, pipes were set in place at the location of the holes and the holes were drilled through these pipes with a diamond drill, after a sufficient depth of concrete had been placed around them. The practice was then adopted of drilling and grouting a hole before the next one was drilled, but, on account of the tight condition of the rock, there was very little difference between the results obtained by the two methods.

The grouting machine was of the same type as that used at Estacada. The cement was a 45% blend of sand-cement (45% of granite and 55% of Portland cement), ground to such a fineness that 90% would pass the No. 200 sieve. The grout was mixed in proportions of about 1 part cement to 5 parts water, except in a few cases, when it was thickened up to about a 1:3 mixture. Pressure for the leakage

test and for final application on the grout, was obtained from a tank set at an elevation of about 15 or 20 ft. above the level of full reservoir. Air for the operation of the grouting machine was obtained from the main compressor plant, and gave a pressure of about 95 lb. at the machine, which, however, was set at an elevation considerably above the bottom of the hole, so that the pressure at that point was in the neighborhood of from 150 to 160 lb. for the lowest holes. Mr. Paul.

Before making the leakage test, immediately prior to grouting, each hole was carefully washed out and sounded; then the leakage test was applied from the elevated tank. The tank was graduated to indicate gallons, and the leakage test was applied to each hole, under full pressure, for a period of at least 10 min. As was expected, most of the holes showed practically no leakage before grouting, and took only about enough grout to fill the hole. In a few cases, however, an appreciable leakage was detected, and some grout was forced into the rock, although up to date the maximum quantity of cement forced into any one hole has been only about 3 bbl. To practically all the holes full air pressure was applied at the beginning, but, in a very few instances, it was found more advantageous to begin with a pressure of about 25 lb. and increase this gradually to the maximum as the hole tightened. When a hole would take no more grout, the water pressure from the tank above the dam was applied—with the idea of driving the grout home—with a pressure in excess of that which would develop from a full reservoir. At the time of this second application of water pressure, careful measurements were taken at the tank in order to make sure that the hole was tight before removing the grouting apparatus.

As to the success of the operations at Arrowrock, there is direct evidence in at least two unrelated instances that the grouting is accomplishing the results expected. At one point, near the bottom of the north abutment, where several adjacent holes showed an appreciable leakage before grouting, it was decided to drill and grout some additional holes before the pumps were pulled and the water allowed to rise to the river level. These holes were drilled with a diamond drill, from just outside the up-stream face of the dam, and were given a slight inclination, so as to bring the bottom of the hole well under the dam and in the region covered by the previous grouting operations. They were spaced so as to pass between the holes already grouted. Most of these holes were drilled to depths greater than those of the original holes, and each hole was grouted before the next one was drilled. Without exception, it was found that these holes were practically tight when the leakage test was applied, proving without question that the earlier grouting operations had tightened up the foundation successfully.

In another case, in excavating the keyway up one of the abutments, some seamy rock was encountered, which had been affected by previous grouting operations, and in the excavation of this rock, it was noted

Mr. Paul.

that the seams were completely filled with grout. In some cases, cracks which otherwise could hardly have been detected, could be followed by the exceedingly thin line of grout that marked them, resembling more than anything else a fine, sharp, pencil line. This grout was encountered at an elevation of at least 60 ft. above where the grouting machine had been located, showing that the grout, in this case, at least, had traveled for a long distance and had filled the seams very thoroughly.

As to the general success of grouting operations, it is believed that so much depends on local conditions that it is impossible, in any given case, to predict just what the results may be or how far they may be relied on. It seems to the writer that a hard rock with fairly continuous seams presents ideal conditions for grouting operations, as long as the surface seams may be sealed off so that the grout may be forced down and out, rather than up to the surface, where it can escape freely. Loose gravel or sand is the other extreme, and it is very doubtful if such material can be grouted successfully. In fact, the writer has not yet heard of an instance where this has been done. All grades of material between the two may be encountered, in which grouting will give more or less satisfactory results.

The writer is of the opinion that grouting, as a precautionary measure, is excellent treatment for the foundation of any dam of considerable size, but where the safety of the structure depends on the success of the grouting operation, it would be unsafe to go ahead with construction until after grouting had been tried and proven effective. In other words, where grouting is actually needed, it is unsafe to depend on it, except after convincing evidence, at the site, that it will give the results desired.

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PAPERS AND DISCUSSIONS

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AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

Discussion.*

By John C. Koch, Assoc. M. Am. Soc. C. E.†

Mr. Koch.

John C. Koch, Assoc. M. Am. Soc. C. E. (by letter).—Mr. James seems to be under the impression that the writer's investigations and studies of sand-clays were confined to those of a restricted area in North Georgia, and that the sands prevalent were "the Orangeburg sands of the Lafayette formation". For the purpose of making clear that typical sand-clays, representative of the entire State of Georgia, were examined, it may be sufficient to state that analyses were made from numerous samples from 23 counties lying in the Coastal Plain, from 5 counties along the Fall Line, and from 20 counties in the Piedmont Region. The writer has not been able to find any geological account of the existence of the Orangeburg sands of the Lafayette formation in the northern third of Georgia, which is almost altogether of igneous and metamorphic character and is often referred to as the Crystalline Area. According to Professor S. W. McCallie, State Geologist of Georgia, the Lafayette formation is found only on the Coastal Plain, and has probably been confused with certain red sands and gravels of the Eocene and Cretaceous formations, also occurring there. As is generally known, the sand-clays of the Crystalline Area give far better results in road work than those of other formations, the sands of the Lafayette formation being only of minor importance.

The road problem of the Coastal Plain along the Atlantic seaboard is how to overcome the difficulties incident to a surface soil composed largely of very fine sand. It is often possible to find a fair supply of clay a few feet below the surface. This may contain from 20 to

* Continued from May, 1914, *Proceedings*.
† Author's closure.

Mr. Koch.

60% of fine sand. If such material is used, it generally produces fair results, and is certainly much better than the sand road. With reference to the case where the sands are floury and the clays run high in fine-grained sand, an alleviation of sand-road conditions may easily be made with this material. The writer, however, would hesitate to present this as a desirable or satisfactory criterion for sand-clay roads in general.

The simple field tests described by Mr. James, in mixing sand and clay, are applicable, of course, only to cases where artificial mixtures are to be made. These and similar tests have been considered by the writer, but his experience has been that small spheres of the same mixture gave widely variable results, due to different percentages of moisture held in the material. By heating the spheres on a sand bath at 100° cent. until they are of constant weight, the results will be more uniform.

Mr. Blanchard's suggestion that more detailed information relative to the traffic on the various types of roads discussed in the paper would be of value to road engineers, is concurred in by the writer. Probably no subject in road literature is more difficult of adequate statistical treatment than that of traffic. A traffic census will show great fluctuations in weekly as well as monthly averages, and a census taken before and after road improvement will usually show a greatly increased traffic after such improvement. The improvement of a road often increases the traffic on that road without, apparently, affecting other roads adjoining it.

TABLE 5.—TRAFFIC ANALYSIS OF SAND-CLAY ROADS.

(Approximate.)

Type of vehicle.	AVERAGE DAILY NUMBER OF VEHICLES (BOTH WAYS).											
	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Horse-drawn, total	144	216	261	179	144	144	144	170	288	288	179	104
Buggies	20	30	35	25	20	20	20	25	40	40	25	20
One-horse wagon	60	90	100	75	60	60	60	70	120	120	75	40
Two-horse wagon	60	90	120	75	60	60	60	70	120	120	75	40
Four-horse wagon or heavy machinery	4	6	6	4	4	4	4	5	8	8	4	4
Motor-driven, total	20	20	30	40	40	55	55	40	57	57	38	20
Two-passenger automobiles	10	10	15	20	20	30	30	20	30	30	20	10
Four-passenger automobiles	6	6	8	10	10	15	15	10	15	15	10	6
Six-passenger automobiles	2	2	4	6	6	8	8	8	8	8	6	2
Trucks	2	2	3	4	4	2	2	2	4	4	2	2
Ratio, motor to horse-drawn	0.14	0.09	0.11	0.22	0.28	0.38	0.38	0.23	0.20	0.20	0.21	0.19

Table 5 is presented in order to give an idea of the average traffic conditions prevailing on the sand-clay roads of Georgia. It shows

the daily average number of vehicles going to and from a typical county seat or shipping point, over a single road, during each month. Traffic to and from larger cities, of more than 20 000 population, is not considered in this table. On some of the sand-clay roads, the traffic will average less than that given in Table 5, and on about 20% of the mileage the number of vehicles of each class may be three times as many. These figures are based on traffic studies made in many counties of Georgia, and are substantially correct in so far as it is possible to prepare average figures from such data. The ratio of motor-driven to horse-drawn vehicles will undoubtedly steadily increase. Mr. Koch.

The following memoranda concerning the actual loads carried may be of interest in connection with Table 5:

One-horse vehicle, average net load,	750 lb.
Two-horse " " " "	1 500 "
Four-horse " " " "	3 000 "
Motor trucks, " " "	1 500 "

Climatic conditions in Georgia vary considerably, from the southern portion, not far above sea-level, with a very mild climate, to the northern boundary, in the Blue Ridge Mountains, at an elevation of from 600 to 1 500 ft. above sea-level, with a much more rugged climate. Generally, the inferior natural sand-clays are found in the southern part of the State, where the climate is very mild. As a rule, the best and most durable sand-clays are found chiefly in the northern half of the State. In the latter section the winters are not severe, but the ground freezes and thaws all through the winter, so that the tendency to disintegrate the sand-clay surface is possibly as severe in the aggregate as the spring thaw in the more severe climate of the Northern States.

The writer has examined sand-clay road surfaces during the winter, and has studied the effects of the alternate freezing and thawing, for the purpose of arriving at some basis for determining their suitability for withstanding severe freezing and thawing. On the better sand-clay roads the effect of weeks of freezing and thawing was to produce a thin layer of slushy material, from ½ to 1 in. thick, below which the mixture was quite firm and impervious. In a number of such instances, the writer has cleared off the slushy material and carefully cut down through the surfacing which was almost always found to be very dense and hard and to contain scarcely any more moisture than in summer. Where mixtures of the type classified as "Hard" by the writer (in Fig. 1) were used, on removing the surfacing as described, the roadbed beneath the sand-clay was found to be as dry as it would have been in midsummer, if protected by the sand-clay surfacing. This seems to prove that impermeable mixtures should be

Mr. Koch.

used to protect the sub-grade properly, in severe climates. In making examinations of these road surfaces, areas several feet square were taken along the center line as well as in the wheel tracks, so that, as far as possible, freak conditions were eliminated.

Another point which Mr. Blanchard raises is well worthy of further study, namely, the effect of different percentages of sand-clay mixtures retained on the 10-mesh sieve, when such mixtures are used under various climatic and traffic conditions. The writer has not had opportunity to study a sufficient number of such cases to feel warranted in stating any conclusions at this time. It is safe to say, however, that, in a sand-clay mixture, the greater the proportion of material coarser than that retained on a 10-mesh sieve, the greater its durability and the more suitable its use for service in a very cold climate, provided, of course, that the coarse material is firm and hard and similar to quartz or other hard rock.

The writer agrees with Mr. Blanchard that many highway engineers will disagree with him in dividing gravel and sand on the basis of the material retained on or passing the 10-mesh sieve. The writer has used the 10-mesh sieve arbitrarily as the dividing line between sand and gravel. The following is the definition* of gravel as officially adopted by the American Railway Engineering and Maintenance of Way Association:

"Gravel is defined as small worn fragments of rock occurring in natural deposits, that will pass thru a 2½-in. ring and be retained upon a No. 10 screen."

In the writer's experience in testing sand-clays, the 10-mesh sieve has been very satisfactory as a basis for separating the coarser material from the finer. If the 4-mesh sieve had been used in analyzing the coarser materials, probably not more than 1% of the samples examined would have left any gravel on this sieve. Practically all the material classified by the writer as gravel was larger than 10-mesh size and smaller than 4-mesh size, with the single exception of Sample No. 223. This sample contained 34% of material coarser than 10-mesh size, and one-half of this was larger than 4-mesh size.

Mr. Stewart mentions the application of hot oil to the roads of the Parkway Systems of Greater New York, which were built of Hudson cementitious gravel. Similar treatment has been proposed for sand-clay roads in certain counties of Georgia, but it has not been attempted on account of the relatively great expense. The cost of a seal coat of hot oil is as great as the total cost of the sand-clay surfacing, and the only advantage that could be claimed to accrue from the additional expenditure would be a slight diminution of the dust from automobile travel. The quantity of dust raised on sand-clay

* American Civil Engineers' Pocket Book, p. 169.

roads by automobiles depends largely on the character of the construction. Well-constructed roads of a good class of this material yield from one-fifth to one-tenth of the dust that is raised on a water-bound macadam road under the same traffic conditions. Mr. Koch.

Mr. Owen's experience with artificial mixtures of sand and clay illustrates a common difficulty in making such mixtures. If natural sand-clay mixtures are available locally, but are not suitable for use alone, because of a deficiency in either the sand or clay content, this defect may be corrected by the addition of the proper quantity of sand or clay. This method of treatment will generally be found to be more economical than an attempt to build up an artificial mixture by using sand as a basis and adding the proper quantity of clay and mixing. Occasionally, it may be possible to combine two natural mixtures in which an excess of sand occurs in one and an excess of clay in the other. To determine the proportions in which such materials should be used, one may make analyses of the materials and plot curves showing their composition. Usually an inspection of these curves will suggest approximately the proportions in which the materials should be used to obtain the best results. Trial mixtures of the two materials may be studied by plotting the analyses of such mixtures, using as a criterion by which to judge their value, the analyses of sand-clays of known value. Fig. 1 has been used in this manner by the writer with excellent results.

The advantage of using a natural sand-clay as the basis for artificial mixtures lies in the fact that the two materials are somewhat thoroughly mixed, and the incorporation of the additional sand or clay required will be much more easily effected than where none of the mixing has been previously performed by Nature. In making an artificial mixture, using sand as a basis and adding the requisite quantity of clay, the latter forms in clods and balls, and thorough incorporation of the two is accomplished only by a long-continued process of puddling.

The effect of climatic conditions on sand-clay roads has been suggested by Mr. Blanchard and Mr. Owen. In regard to the effect of the severe weather conditions of the Northern States, the writer has no definite information. The following mileages of sand-clay roads were reported* in 1909 for the States named:

Connecticut	1 214
Delaware	6
Georgia	4 326
Idaho	398
Iowa	575
Kansas	202

* *Bulletin* No. 41, U. S. Office of Public Roads, p. 42.

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Michigan	2 381
Minnesota	1 051
Oregon	345
Rhode Island	6
South Dakota	129
Washington	1 223
Wisconsin	1 013
Total, leaving out Georgia.	8 543

From the foregoing, it appears to the writer that there is a considerable mileage of roads of this type in use in the Northern States, and that since 1909 there should have been developed among road builders in the States named a large amount of valuable information regarding the use and possibilities of sand-clay road construction. The total mileage of such roads in the United States reported for 1909 was 24 601, so that more than one-third of the entire mileage reported in that year was in what may be called the Northern States.

The writer wishes to call attention to the experiences of several counties, that are of interest because of the great contrast they afford in the economy of sand-clay road construction as compared with water-bound macadam. Therefore, the following examples are presented for consideration.

Habersham County, Ga.—A macadam road was laid in 1909, if the writer has been correctly informed, at an expense of about $2 000 for a length of 2 000 ft. This road was 25 ft. wide, and had field-stone curbs. The macadam was about 5 in. thick after rolling. In 2 years the road had become so rough as to be avoided by the traffic whenever the parallel side streets were not too muddy. In 1912 a new road was built of a natural sand-clay mixture, extending from the end of the old macadam section mentioned, to the railroad station at Clarkesville. This was curbed with field stone, and cost $1 200 per mile. It is smoother after a year's service than the macadam road ever was, and is easily kept in excellent repair on account of the large quantity of good sand-clay close at hand.

Hall County, Ga.—In the period, 1905 to 1908, this county established quarries, installed machinery of all sorts for road-making, and spent about $40 000 in building water-bound macadam roads. The rock used was a local gneiss. The roads, for the most part, were surfaced 5 in. thick and for a width of 20 ft. The total length actually constructed was between 8 and 10 miles, as nearly as the writer can learn. The total road mileage of this county is about 900, of which probably 200 miles would serve 50% of the traffic. From this it can be readily seen how impossible it would be to construct the most important roads with macadam. The average annual income from taxes for road improvement in this county is about $15 000.

Mr. Koch.

In the fall of 1911 almost all the macadam roads which had been constructed were in such bad condition that reconstruction seemed absolutely necessary. To reconstruct these roads with macadam was a sheer financial impossibility, as the county was still in debt for the original construction and equipment. Temporary repairs were made, which rendered the roads passable that winter. In the fall of 1912, the writer was consulted by the county authorities as to what could be done to improve road conditions, and as to the adoption of a definite low-cost programme that was financially feasible. After an examination of the macadam and other roads, all of which were in wretched condition, especially the former, the writer recommended the adoption of the sand-clay road wherever that material could be used without excessive expense. After sampling a number of large areas of natural topsoil sand-clay, the writer found that, with few exceptions, excellent natural mixtures were readily available with short hauls; so that almost all the important roads were favorably situated for reconstruction with these mixtures. It was also found that the deep holes in the macadam roads could be filled with the sand-clay, which bonded well with the macadam, and, at comparatively small expense, many of these roads were thus repaired. The remainder was plowed up and loosened and then a 10-in. layer of sand-clay was placed over it and compacted by the traffic. Since then the county has carried out a consistent policy of repairing old macadam with sand-clay until the macadam is worn out, and making all new construction of sand-clay. Under this new policy, 20 miles of new sand-clay roads have already been constructed, at an average cost of about $425 per mile; the old macadam is still giving some service, and the improved roads are passable at all seasons of the year. All the expenses incident to this new policy have been met by current revenues.

Clarke County, Ga.—The experience of this county was somewhat similar to that of Hall County, except that sand-clay construction has been in use here for nearly 7 years. During the period, 1905-08, Clarke County spent about $50 000 on the construction of 7 or 8 miles of water-bound macadam roads. This had been accomplished in a little more than 3 years. Then it became evident that the cost of maintenance was a far more serious item than had been thought. It was so great that it soon became clear that macadam was too expensive an investment for the county's revenues.

In 1907, Mr. W. H. Holman, Superintendent of County Roads, began experimenting with natural mixtures of sand-clay, taken from the surface of cultivated fields, as a road-surfacing material. This material is often referred to as "topsoil". The experiments were so successful that topsoil was soon adopted as the surfacing material for all roads, and by 1913, practically all the important roads of the

Mr. Koch.

county had been surfaced with it, for a total length of 100 miles. This large mileage was built with current revenues (approximately $20 000 annually), without a bond issue or the assumption of a heavy burden of taxation. This splendid work has been brought about largely as a result of the untiring efforts of C. M. Strahan, M. Am. Soc. C. E., who, as County Engineer, has been identified with this road work for many years.

The average annual cost per mile of the macadam roads built by this county, including bond interest, sinking fund, and maintenance, was found to be about $1 200 (neglecting the fact that after from 4 to 6 years of service they were covered with topsoil to prevent complete raveling and destruction).

The cost of sand-clay roads in this county has averaged about $500 per mile, so that the annual cost of 1 mile of macadam (neglecting depreciation entirely), was sufficient to build 2.4 miles of new sand-clay road. The cost of maintenance of sand-clay roads of good quality is very low, averaging $5 per mile per annum in this county. Poorer qualities of sand-clay can be kept in excellent condition at an expense of from $10 to $30 per mile per annum depending on its quality and the character of maintenance required.

In conclusion, it should be stated that the specific cases just described are presented as concrete examples of typical experiences in many parts of the South, and that these cases are not referred to in any spirit of criticism, but for the purpose of emphasizing the fact that, in many parts of the United States in which the principal, but not the only, difficulty has been poverty, sand-clay offers the only feasible solution of the road problem. There are many counties where the mileage of public roads is so great that there is a mile of road for every three to ten voters. It is especially for such sparsely settled districts that the improved low-cost road is a necessity. Though the sand-clay road is adapted to the heavy traffic of much more densely settled communities than those indicated, it also offers to sparsely settled communities hope of improved roads without an excessive burden of taxation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE

Discussion.*

BY MESSRS. JAMES OWEN AND G. E. P. SMITH.

Mr. Owen.

JAMES OWEN, M. AM. SOC. C. E.—Although Mr. Lee's paper is confined somewhat to underground water supplies in the western part of the United States, a great deal of investigation has been done and a great deal of money has been spent and wasted on such supplies in the eastern sections and in localities near New York City, and it seems that if it were possible in the future to define certain rules and principles governing the question of getting water from the ground, it would be better for everybody.

The section around New York has a variety of topographical and geological conditions. According to the speaker's idea, the underground supplies can be defined under about four heads, that is, the original sandstone deposition, prior to the volcanic upheaval; the volcanic upheaval of gneiss in Westchester County and New York, and the trap in New Jersey; the glacial and post-glacial depositions, and, incidental to these, the terminal moraines; and finally, in lower New Jersey, what might be called the post-tertiary deposition. The first and the last, the original sedimentary deposition of the sandstone and the post-tertiary deposition, have fairly well-defined radii of supply.

It is well known that water can usually be obtained by driving through sandstone. In New Jersey, if one drives through certain strata, water can be obtained, and an interesting incident was shown

*This discussion (of the paper by Charles H. Lee, Assoc. M. Am. Soc. C. E., published in April, 1914, *Proceedings*, and presented at the meeting of May 6th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Owen.

when Asbury Park, N. J., desired a water supply and was advised by Professor Gay, State Geologist of New Jersey 30 years ago, to go down 800 ft., at which depth all the necessary water could be obtained. On trial, water was found within 40 ft. The final result has been that all through that shore territory the supply of water has been ample, good, and economical. In this section, however, the main consumption has only lasted about 4 months in the year, and during the other 8 months, the disposal of the underground supply would have failed to be profitable for steady communities.

In the sandstone formation, a good and reliable supply can always be obtained, subject to the chemical and natural properties of the water. In New Jersey, wells have been sunk 400 or 500 ft. below tidewater and ample water has been found, but its chemical constituents have made it rather detrimental for public use. In one case, where the well was put down 400 ft., the deposition of lime was about 8 in. in 4 months. That, of course, debarred its use for public purposes. The water, however, was storage water, and the slow percolation through the sandstone had not allowed for a free flow; consequently, the chemical deposition ensued.

In the other two regions, the volcanic and the post-glacial formations, there is a certain amount of uncertainty in regard to the supply. In the volcanic formation with a slight cover, of course, there is no water unless one goes into the rock; and in either the trap, gneiss, or Atlantic formation, the supply is uncertain, erratic, and unreliable, and very rarely successful.

In the post-glacial formation, the drifts fill up the subterranean canals, as they may be called, and the terminal moraine depositions. In this case the water supply is fairly reliable within certain limits. In the speaker's experience there have been two or three curious propositions relating to this question. A certain city put in a water plant in a post-glacial deposition. When the wells were put down they overflowed. Pipes were driven down 80 or 100 ft., and left up in the air 10 or 15 ft., and still the wells overflowed. The pumping plant was started, and, of course, the overflow was gradually lowered. Incidental to that was the fact that, a little above the pumping plant, a man owned a very heavy spring which was totally stopped after a short period of pumping. He brought suit against the company, and, by a curious coincidence, the pumping plant broke down at a certain hour on a certain day, and a certain number of hours afterward, the spring ran afresh. That incident showed the underground capacity of that subterranean gulch, including the capacity of the underground storage.

That formation was almost all sand and gravel. The water-table in 3 years of pumping was lowered from an overflow of 6 or 8 ft. above ground to a ground-water flow of 22 ft. below the surface,

showing that the company was pumping beyond the capacity of the delivery or the water flow of the country. Mr. Owen.

Take the other case, where there is a till flow, that is, in the glacial deposit termed the till, where the percolation is very slow. In certain cases under the speaker's care, the flow of the wells after a rainstorm was carefully timed, and the percolation instead of being immediate, took about 2 days. In the case of one large well, where six or seven holes had been bored into the limestone, and where it was important to have supply enough for the examination, careful note was taken of the times when there was enough water and when the supply was deficient. Examination showed that, after a heavy rainstorm, it took 2 days for this flow through the till, that is, the drift deposit, to get through the rock and into the wells. This shows the extremely slow rate of percolation.

There are a great many interesting questions in the whole region about New York, and it is especially necessary to have known facts tabulated, as there has been a great deal of money wasted, especially in developing what is known as a mysterious underground supply.

In the case previously cited, the wells were put in for the city under the direction of a competent engineer, and the speaker remarked at the time that he was surprised at his being brave enough to put in such a plant for that community. The result has been that that city has been compelled to buy up land and territory after territory to provide a sufficient underground supply.

G. E. P. Smith, M. Am. Soc. C. E. (by letter).—The publication of this scholarly paper on investigations in the Owens Valley, and the studies and conclusions based thereon, must be welcomed by all hydraulic engineers of the arid West. In addition to its immediate or local value, the paper affords a clear and concise exposition of the general principles of ground-water storage, of recharge and loss, and of the extent to which ground-waters can be drawn on for municipal water supplies, irrigation, or other purposes. So far as the writer knows, it is the first comprehensive work of its kind, and the author has the distinction of being a true pioneer of the profession. The paper itself marks a new era in the development of ground-water studies, an era in which scientific basis and logical methods supersede much idle speculation and many misconceptions. Mr. Smith.

Too much credit cannot be given to Mr. Lee for the thoroughness and application of system displayed in his investigations. These features are exemplified in the excellent analyses of rainfall distribution, of seepage losses, and of soil evaporation tests. Where arbitrary factors have been necessitated, they have been based on painstaking inspections and on rare good judgment. In such cases, also, the author was assisted by the extreme uniformity of natural conditions in the Owens Valley, especially of topography, valley fill, soil and

Mr. Smith.

vegetation. The application of system in the investigational work is reflected in its orderly presentation, and in the use of graphical methods, features which add greatly to the value of the paper.

The writer's purpose is not to criticize, but to broaden the foundation on which the deductions and generalizations of the paper are based, and to assist in the extension of their application to a wider range of climatic and geologic conditions.

Beginning in 1906, the Arizona Agricultural Experiment Station has carried on ground-water investigations under the writer's charge in several valleys of southern Arizona. These valleys vary greatly in the degree of aridity, from the grassy sub-arid Sulphur Spring Valley, which is comparable to Owens Valley, to the severely arid Lower Gila Valley. The chief determining factor is the altitude. The ground-water hydrology is exceedingly diverse in character, so that it is difficult to frame generalizations that are applicable to all the valleys.

The most detailed study has been that of the valley of the Rillito,* a tributary of the Santa Cruz, as shown in Fig. 13. There are striking similarities between it and Owens Valley. Both are desert valleys in which the surrounding drainage is toward a broad, flat area, or playa, and therefore they come within the meaning of the term "bolson" or "semi-bolson", as defined by Tolman.† The Pantano watershed spills its surplus water to the Rillito; the Rillito delivers a portion of its drainage to the Santa Cruz, and the Santa Cruz, on rare occasions, has a continuous discharge to the Gila River. Excluding the Pantano bolson with its 620 sq. miles, the total drainage area of the Rillito is 327 sq. miles, of which 56% is mountainous (granitic), 30% consists of dissected gravelly outwash slopes, and 14% is valley land. The area is nearly equal to that of the Independence Region, though the percentage of mountain area is higher. The lengths of the valleys are the same—25 miles—and both derive practically all their drainage from the right-hand side. The mountain rainfall, also, is equal in the two cases, varying from 10 or 12 in. in the foot-hills to 30 or 35 in. at the crests.

The distinctions are: the Rillito Valley lies at an elevation of about 2 400 ft.—considerably lower than Owens Valley—and consequently the temperature is higher and evaporation is greater; the mountainous area drained by the Rillito is on the windward side, and hence the rainfall is comparatively high for the region; the rain-

* *Bulletin* No. 61, Arizona Agricultural Experiment Station, 1910.

† He states: "I therefore suggest that the word be used to cover the watershed of a centripetal drainage system, including all the area within the limits of the divides. The bolson may depart somewhat from a perfect topographical basin, for evaporation on a slope may prevent the development of a through drainage, and foster the centripetal variety. Those bolsons whose surface water in times of flood reaches some river thoroughfare some lower bolson, or the ocean direct, and consequently the playa portion * * * is poorly developed or lacking, may be called semi-bolsons."—*Journal of Geology*, XVII, No. 2, p. 141, February-March, 1909.

Mr. Smith.

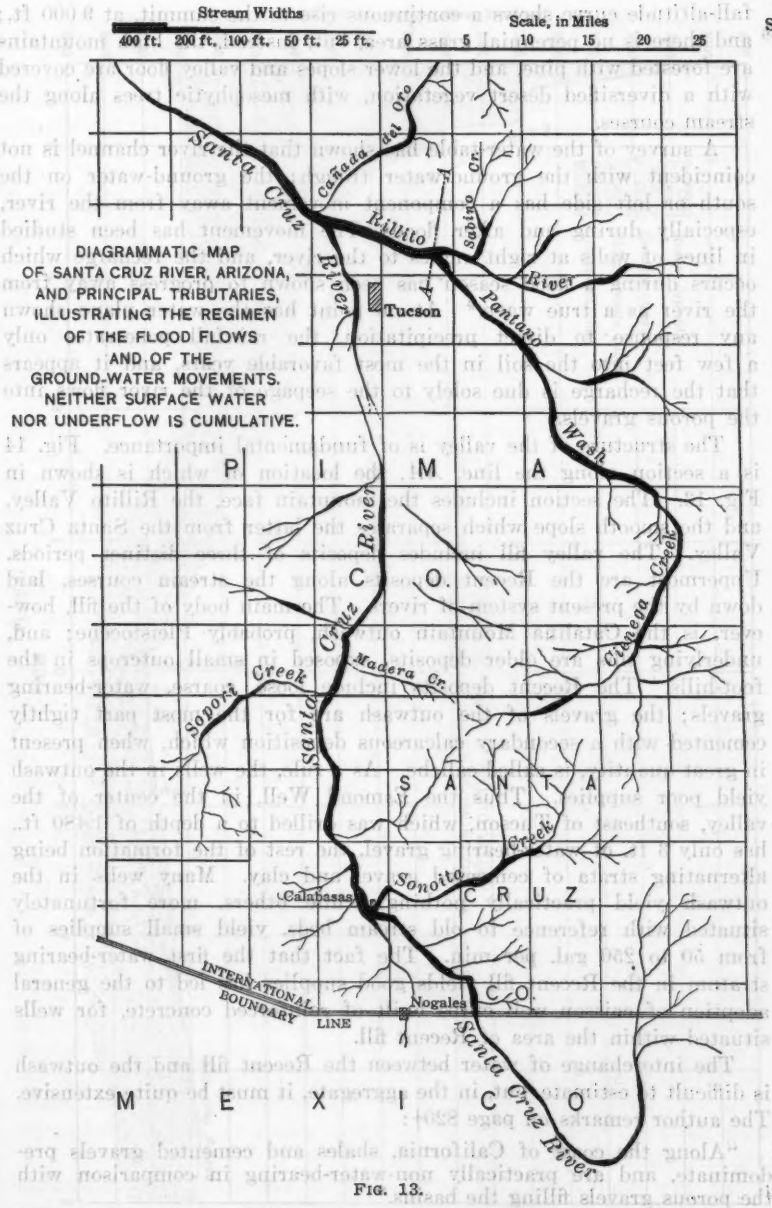

FIG. 13.

Mr. Smith.

fall-altitude curve shows a continuous rise to the summit, at 9 000 ft.; and there is no perennial grass area, but, instead, the high mountains are forested with pine, and the lower slopes and valley floor are covered with a diversified desert vegetation, with mesophytic trees along the stream courses.

A survey of the water-table has shown that the river channel is not coincident with the ground-water trough; the ground-water on the south or left side has a component movement away from the river, especially during and after floods. The movement has been studied in lines of wells at right angles to the river, and the recharge which occurs during a flood season has been shown to progress away from the river as a true wave.* At no point has the water plane shown any response to direct precipitation, the rainfall penetrates only a few feet into the soil in the most favorable years, and it appears that the recharge is due solely to the seepage of the river flows into the porous gravels.

The structure of the valley is of fundamental importance. Fig. 14 is a section along the line, *AA*, the location of which is shown in Fig. 13. The section includes the mountain face, the Rillito Valley, and the smooth slope which separates the latter from the Santa Cruz Valley. The valley fill includes deposits of three distinct periods. Uppermost are the Recent deposits along the stream courses, laid down by the present system of rivers. The main body of the fill, however, is the Catalina Mountain outwash, probably Pleistocene; and, underlying this are older deposits, exposed in small outcrops in the foot-hills. The Recent deposits include loose, coarse, water-bearing gravels; the gravels of the outwash are for the most part tightly cemented with a secondary calcareous deposition which, when present in great quantity, is called caliche. As a rule, the wells in the outwash yield poor supplies. Thus the Esmond Well, in the center of the valley, southeast of Tucson, which was drilled to a depth of 1 480 ft., has only 3 ft. of water-bearing gravel, the rest of the formation being alternating strata of cemented gravel and clay. Many wells in the outwash yield practically nothing, while others, more fortunately situated with reference to old stream beds, yield small supplies of from 50 to 250 gal. per min. The fact that the first water-bearing stratum in the Recent fill yields good supplies has led to the general adoption of caisson well curbs built of reinforced concrete, for wells situated within the area of Recent fill.

The interchange of water between the Recent fill and the outwash is difficult to estimate, but, in the aggregate, it must be quite extensive. The author remarks on page 820†:

"Along the coast of California, shales and cemented gravels predominate, and are practically non-water-bearing in comparison with the porous gravels filling the basins."

* *Bulletin* No. 64, Arizona Agricultural Experiment Station, p. 184.

† *Proceedings*, Am. Soc. C. E., for April, 1914.

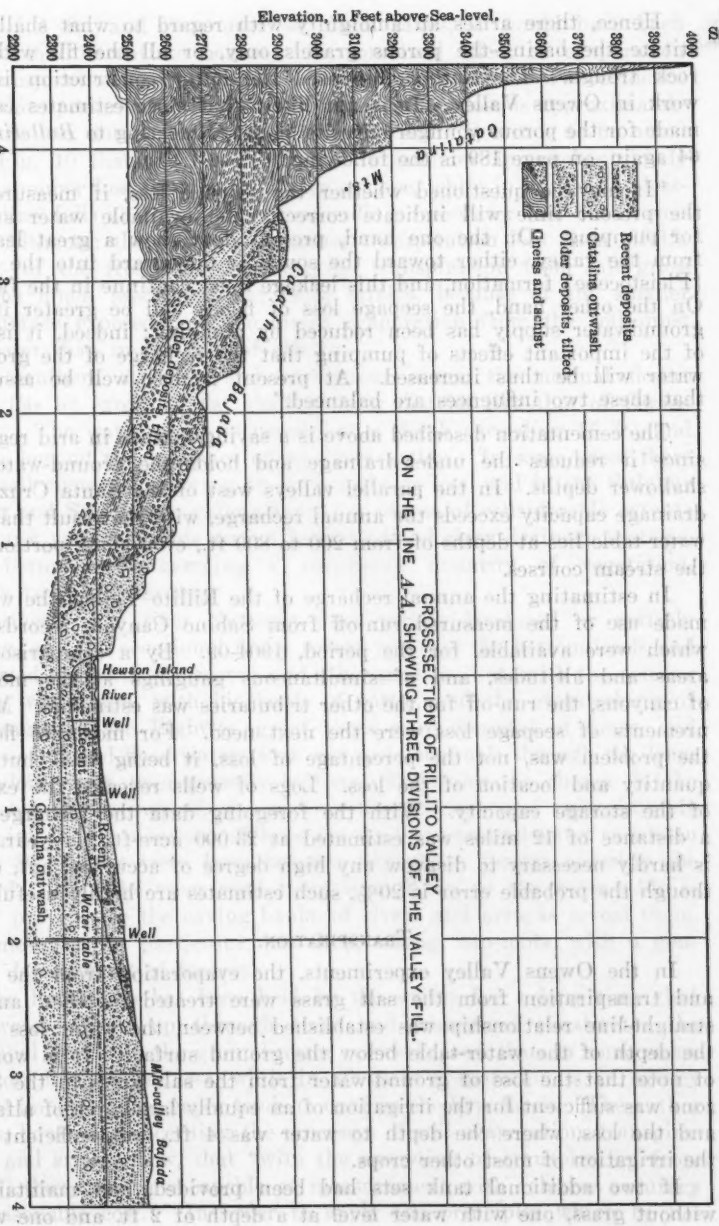

FIG. 14.—CROSS-SECTION OF RILLITO VALLEY ON THE LINE *A-A*, SHOWING THREE DIVISIONS OF THE VALLEY FILL.

Mr. Smith.

Hence, there arises an ambiguity with regard to what shall constitute the basin—the porous gravels only, or all the fill within a rock trough. The author has used the latter construction in his work in Owens Valley. It is not likely that close estimates can be made for the porous younger deposits alone. Referring to *Bulletin* No. 64 again, on page 189 is the following:

"It may be questioned whether the seepage loss, if measured at the present time will indicate correctly the available water supply for pumping. On the one hand, present data show a great leakage from the valley, either toward the south or downward into the older [Pleistocene] formation, and this leakage must continue in the future. On the other hand, the seepage loss of floods will be greater if the groundwater supply has been reduced by pumping; indeed, it is one of the important effects of pumping that the recharge of the groundwater will be thus increased. At present it may well be assumed that these two influences are balanced."

The cementation described above is a saving feature in arid regions, since it reduces the under-drainage and holds the ground-water at shallower depths. In the parallel valleys west of the Santa Cruz, the drainage capacity exceeds the annual recharge, with the result that the water-table lies at depths of from 200 to 800 ft., even along portions of the stream courses.

In estimating the annual recharge of the Rillito Valley, the writer made use of the measured run-off from Sabino Canyon, records for which were available, for the period, 1904-09. By a comparison of areas and altitudes, and of simultaneous gaugings at the mouths of canyons, the run-off for the other tributaries was estimated. Measurements of seepage loss were the next need. For moderate floods, the problem was, not the percentage of loss, it being total, but the quantity and location of the loss. Logs of wells revealed the extent of the storage capacity. With the foregoing data the recharge for a distance of 12 miles was estimated at 23 000 acre-ft. per year. It is hardly necessary to disavow any high degree of accuracy, but, even though the probable error is 20%, such estimates are highly useful.

Transpiration.

In the Owens Valley experiments, the evaporation from the soil and transpiration from the salt grass were treated together, and a straight-line relationship was established between the water loss and the depth of the water-table below the ground surface. It is worthy of note that the loss of ground-water from the salt grass in the 3-ft. zone was sufficient for the irrigation of an equally large area of alfalfa, and the loss, where the depth to water was 4 ft., was sufficient for the irrigation of most other crops.

If two additional tank sets had been provided, and maintained without grass, one with water level at a depth of 2 ft. and one with

depth of 4 ft., and thus used as check tanks, it would have been possible to differentiate between the combined effect of evaporation and transpiration and the effect of evaporation alone. The evidence of the data presented indicates that the combined loss is approximately twice the loss due to evaporation from the soil. For it is to be noted from Fig. 10 that the loss-depth line in summer approaches closely to the summer loss from a free water surface, while in winter the loss-depth line falls short of the corresponding free-water loss about one-half. Although the estimated summer transpiration loss from alfalfa is 87% of the free-water loss, yet the transpiration from salt grass must be much less than that from alfalfa, because of the smaller leaf surface exposed, and the direct evaporation from the soil must be greater than in the case of alfalfa. Mr. Smith.

In southern Arizona and contiguous areas, the normal desert valley has no extensive flat shallow-water district like that of Owens Valley. The water plane is usually at a depth exceeding 15 ft., and is thus beyond the influence of capillary action. In some cases there are small cienegas, covering an acre, more or less, and in the Sulphur Spring and San Simon valleys there are playas of considerable extent; but many valleys of this region have modified river drainage systems, with bottom-lands carrying a surprising quantity of vegetation, usually trees and shrubs.

Formerly, the river channels were poorly developed or entirely lacking, and the occasional floods spread out over the bottom-lands and supported a growth of sacaton and other grasses; but, with the coming of the white man, with his herds of cattle, and the overstocking of the ranges in the Eighties, great areas were denuded of grass, and the concentrated flood-waters soon cut wide channels through the loam and adobe down to a gravelly bottom. Following the change in the character of the run-off, the sacaton disappeared, and trees, notably mesquite, took possession. Where the depth to ground-water is within 25 or 30 ft., the trees have become continuous forests covering the ground. That the trees send their roots down to the water-table is easily proven, for the caving banks of rivers and arroyas reveal them. The mesquite, in particular, has deep, strong, tap-roots, with a generous development of feeders.

The hypothesis has been held by the writer for a long time that the water drawn up through trees and transpired constitutes the principal loss from the ground-water reservoir, and that, in some cases, this loss is the total loss, while in all cases evaporation is an agency of less import. This paper tends to confirm the hypothesis. The author states, however, concerning the processes of soil evaporation, transpiration, and stream flow, that "with the exception of transpiration from trees", they "are now capable of measurement with relative accuracy at reasonable cost." In order to apply the principles which he has

Mr Smith.

so ably developed to the regions somewhat more arid than Owens Valley, it is essential to learn much more than is now known about tree transpiration.

Botanical literature offers little assistance in this problem. The single experiment or estimate which is found in all botanical textbooks is that in Austria a high beechwood forest transpires 30 000 liters daily per hectare during the growing season of 6 months. This is equivalent to 22 in. depth of water over the land. It is stated, further, that from 250 to 500 grammes of water are transpired for every gramme of dry solid matter produced.

Mr. Lee says of transpiration that it "differs in different species of plants," and "King's experiments indicate that humidity does not affect transpiration. For a species growing in a definite locality, light and available soil moisture are the controlling factors." These statements are surely open to question, as will be explained presently. It is probable that all plants tend toward the same rate of relative transpiration per unit of surface exposed, and the principle is well established that humidity is a potent factor of transpiration, as well as are light and soil moisture. The significance of these corrections lies in the fact that ground-water reservoirs are of chief importance in arid regions; and in such regions, with low humidity, brilliant light, and high rates of transpiration, even the xerophytic, or drouth-resistant, plants, under certain conditions, become most profligate in dissipating the only available water resources.

Recent researches of the Desert Botanical Laboratory of the Carnegie Institution have brought out some pertinent new principles of transpiration. The work of F. Shreve in the tropical mountain rain-forest of Jamaica has yielded the conclusions that humidity is a factor of much influence, and that transpiration is approximately proportional to evaporation. He says:*

"Although high humidities (90 to 95 per cent.) have been found to reduce the absolute rate of transpiration below its amount at relatively low humidities (55 to 71 per cent.), as is to be expected, the rate of relative transpiration continued to be of the same general order of magnitude at all humidities which are well above the minimal point for rain-forest plants."

Relative transpiration is a term used to define the ratio of transpiration to evaporation. Further, Shreve has studied data obtained by him in Jamaica in comparison with similar investigations carried on at Tucson, by B. E. Livingston on several desert ephemerals, and by Edith B. Shreve with the palo verde, *Parkinsonia,* which is rated as "a most successful desert tree". To quote again:†

"A general review of the data under comparison indicates that, in spite of minor differences, there is a greater uniformity among

* "Year Book No. 12," Carnegie Institution of Washington, 1913, p. 74.
† *Ibid*, pp. 75-76.

the relative transpiration maxima for the rain-forest and for the desert than might be expected. When such a uniformity is considered in the light of the fact that the evaporation is very many times greater in the Arizona desert than in the Jamaican rain-forest, it forces the conclusion that the transpiration-rates in the plants of the two regions must be roughly proportional to the evaporation-rates, else the relative transpiration-rates would not remain so nearly equal. In short, it is the desert plants in which the rate of transpiration is high and the rain-forest plants in which it is low, which is quite the reverse of the commonly accepted view." Mr. Smith.

G. F. Freeman, plant breeder of the Arizona Agricultural Experiment Station, has made simultaneous tests of transpiration on a peach tree and a creosote bush growing in close proximity. He found a very slightly higher transpiration-rate for the peach tree per pound of green matter, but, when based on the area of leaf surface, the rate for the creosote bush was in excess.* The creosote bush is an extremely xerophytic plant, and forms the principal vegetation of the dryest slopes.

The establishment of the principle of equal relative transpiration-rates makes it possible to eliminate from many discussions factors such as light, air pressure, temperature, and humidity, and to base estimates of transpiration on the more easily measured evaporation from a free water surface. Also, the principle harmonizes the author's estimate of annual transpiration loss from alfalfa with King's estimate for clover in Wisconsin. The leaf characteristics of the two crops are very similar. The estimate for alfalfa is 41 in. depth of water transpired through the plants and 10 in. additional evaporated from the soil; for clover the estimate is 22.3 in. for both processes combined. The rates of evaporation in Owens Valley and in Wisconsin, however, are approximately as 2 to 1.† Since the rates for southern Arizona and for Wisconsin are as $2\frac{1}{2}$ to 1, the inference can be drawn that in Arizona it requires about 56 in. for the proper irrigation of alfalfa.

Before condemning all desert vegetation indiscriminately, however, an ameliorating factor must be taken into consideration. Desert plants possess many queer habits by which they protect themselves from the tendency toward high transpiration-rates. On hot dry days they close their stomatal openings. In times of drouth they drop their leaves. The leaves are invariably small. The leaves of many species are covered with hairs. Some of the cacti are provided with water storage organs either in the roots or in the stems. Hence, plants pass successfully through seasons when the soil moisture around their roots becomes as low as 4 per cent. This power of self-protection is vital to the vegetation of the outwash slopes (usually but improperly called mesas), where the depth to the water level ranges sometimes

* Unpublished work.

† See new Evaporation Chart, "The Plant World," Vol. XIV, No. 9, 1911, p. 219.

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as great as 600 ft. There is no evidence, however, that the power is exercised by trees growing on the bottom-lands, where the roots are bountifully supplied from below the ground-water table, or by alfalfa and other field crops, so long as they are well irrigated.

Adopting now the principle that transpiration varies as evaporation, soil moisture conditions being assumed to be uniform, the transpiration-rate for the beechwood forest previously quoted must be multiplied by $2\frac{1}{2}$ to obtain the rate for arid regions. The result is 55 in. depth of water per annum, a quantity that probably represents fairly well the transpiration loss from the cottonwood trees which fringe the rivers for miles and are abundant usually on the upper courses of the tributaries. The loss from mesquite forest, on account of the smaller leaf expanse, is perhaps one-half as great as that from cottonwood trees.

As a corollary to the foregoing discussion, it is clear that the duty of water to be provided for in any locality is proportional to the evaporation-rate. On this basis, the high duty of water maintained in the San Bernardino Valley, in southern California, 7 or 8 acres per miner's inch, is no more creditable than the duty in Salt River Valley, Arizona, where for the last 3 years, the average duty of water, delivered, has been 5.4 acres per miner's inch. Following the over-use of water in many localities where it is abundant, some irrigationists now go to the other extreme, and advocate an impossibly high duty in the most desert valleys. Investigations relating to duty of water should be accompanied always by current observations of the evaporation rate, so that the results, when published, may be of more than local value. This precaution for making the investigations of wide interest has seldom been exercised.

A Principle of Ground-Water Hydrology.

That the ground-waters in arid valleys are not cumulative, is a principle which needs additional emphasis. Although there is a continuous movement of ground-water longitudinally in a valley, yet, in the main, the ground-water of one region does not get down into the next region. In this respect there is a close analogy between surface flows and underflows. Fig. 13 is a diagrammatic map of the Santa Cruz River and its principal tributaries. The widths of the lines are roughly proportional to the widths of the river channels, and indicate the regimen of the floods—the narrow torrents of the mountain canyons, and the spreading out over sandy beds and rapid absorption by percolation after leaving the canyon mouths. Normally, the river beds are dry. Many floods of the headwaters do not reach Calabasas, and few floods which pass Calabasas reach Tucson. The river is ever a dwindling stream, and, though draining greater areas, brings less water and smaller floods to the junctions with some of its tributaries.

Mr. Smith.

than do those tributaries. Finally, at a point about 10 miles beyond the limit of the map, the channel is narrowed down to a width of 12 ft., and 2 miles farther on it entirely fades out. Likewise, the ground-water movements are represented by the same map. These movements are most active where the river beds are widest, where the recharging occurs; but at all points along the stream courses the moving ground-water is sustaining a loss, due mostly to transpiration, and the loss in any region is commensurate to the recharge in that region. The factors of recharge, loss, and forward movement are closely interrelated, though of varying importance in different localities within a region in their effects on the ground-water supply.

A logical sequence of these conditions is that extensive concentrated ground-water development is impossible. Vast areas of fertile land that could be reached easily by canals will ever remain desolate, but there are hundreds of small areas, usually stretching along the river courses, capable of reclamation by the utilization of ground-waters. The best of these small projects are to be found farther up stream. "The mountain water will not come down into the desert very far; we must go toward the mountain. The water must be used as near as possible to its source."*

Another sequence of the foregoing principle is that new diversions of ground-water in arid valleys are not so prejudicial to older rights as has been assumed oftentimes, and the doctrine of priority now applied to underflow streams of a definite character needs modification. In a discussion of the application of the doctrine to underflow gravity ditches,† the writer has suggested four points of limitation. They might well be applied to pumping operations also. The limitations relate to the burden of proof, to the extent of the injury, to co-operation in development work, and to the efficiency of the collecting agencies. Another consideration is that active interference between underflow ditches or between wells does not begin at once, but may require many months of lowering water plane, and, before the time has elapsed, one big flood may refill the gravels, whereupon all effects of the previous drafts on the ground-water supply are obliterated.

Estimates of Safe Yield.

The large items of the author's estimates carry much conviction as to their accuracy. Some of the small items, however, possibly, need additional study.

1.—Percolation from Precipitation on Intermediate Mountain Slopes.—The Sierra slopes are said to be of unfissured granite covered by a mantle of loose rock. Table 16 gives the average annual rainfall as 12 to 20 in., derived mainly from the slow rains and snows of winter. Similar conditions exist on the slopes of the Catalina and

* *Proceedings*, 16th National Irrigation Congress, 1908, p. 206.

† 22d Annual Report, Arizona Agricultural Experiment Station, 1911, p. 570.

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Rincon Mountains, near Tucson, but observation indicates that most of the precipitation is absorbed by the porous overburden, wherein it supports an extensive desert vegetation. There is, of course, a slow creeping of the water downward at times, but very little of it reaches the valley. There are springs at the base of the slopes, but they are small, more suitable to be measured in cattle drinks than in second-feet. The run-off factors used for the intermediate mountain slopes of Owens Valley appear to be too high.

2.—The Upper Seeley Spring.—The location of this spring, at the north base of Charlie's Butte, raises a question as to the derivation of its water. If it is lateral flow from the outwash slopes, it is included rightly in the summary of ground-water losses, but if it is river underflow brought to the surface by the intervention of the butte—a hydrologic condition of common occurrence in the Southwest—then it should be omitted from consideration.

3.—It is stated that on the outskirts of the salt grass there is a strip of greasewood and bunch grass, that bordering this there is another zone of luxuriant sagebrush, and that the depth to water beneath these areas is from 8 to 20 ft. In the light of the foregoing discussion on transpiration, it is evident that the water loss from these bordering zones is of so much importance that an effort should be made to measure or estimate it.

4.—Another question of doubt to the writer is whether the underflow at Alabama Hills can be disregarded. The river bottoms here are 3 miles wide, and the rock trough is shown to be at least 832 ft. deep. The slope of the water-table is 8 ft. per mile. Although the valley fill on the Inyo Mountain side is composed of fine sand, yet on the opposite side there may be strata of good water-bearing gravels deposited by living rivers from the Sierra Range. A forward movement of the underflow of 6 in. a day throughout the section implies an item of loss of more than 20 sec-ft. This problem gives the writer more concern because in the arid Arizona valleys the bottom-lands do have usually some good water-bearing gravels. Thus, in the Santa Cruz "narrows", opposite Sentinel Hill, at Tucson, the underflow in the first water stratum must have been at least 10 sec-ft. before the recent development was made. The underflow loss can be measured or estimated by the Slichter method, and should be included in the balance sheet. Perhaps a fair estimate for Owens Valley is that the underflow gain at the north end of the Independence Region equals the underflow loss at the Alabama Hills.

In contrast with the author's orderly estimates are the crude methods which have found favor heretofore. A recent report on ground-water resources in an Arizona valley is based on absurd hypotheses, yet, on account of its authorship, the report ought to have been final and conclusive. The report first recites the water-shed area and rainfall, then applies Newell's run-off curves (which give values

too high for arid regions), then deducts surface run-off, the quantity already applied to irrigation, and an allowance for evaporation from the dry river bed, and asserts that the remainder is underflow at the given cross-section of the valley. It ignores the largest factor of ground-water loss—transpiration—and assumes that the underflow is cumulative from the sources of the stream to the place under consideration. The magnitude of the underflow thus computed is excessively high, and is calculated to invite unwarranted expenditures in development. Mr Smith.

On page 822* is the statement:

"There are * * * two possible methods of measuring the rate of recharge, either by determining the total percolation from various sources into the porous material of the basin, or by determining the ground-water losses."

The latter method is applicable in valleys where the loss areas are fairly compact and the vegetation fairly uniform; but, with a native vegetation of trees and shrubs, diverse in leaf expanse, in height, and in stand, and extending over irregular areas, there is little hope of approximating reliable estimates. On the other hand, as the recharge in very arid valleys is practically all from stream flows, the first method promises more accurate quantitative results. The essential features of the first method are an area survey and some gauging stations carefully selected, at the mouths of representative canyons, at the mouths of representative washes draining the outwash slopes, and near the outlet of the region. Fluctuations in wells, and water contour maps, present excellent qualitative studies, inasmuch as they reveal the direction of ground-water movements, the localities of active recharge, and the areas of loss. The first method has the disadvantage of requiring more attention to the residual losses in determining the safe yield than does the second.

The author has not discussed the probable variations in safe yield from year to year, though this is of great importance. Unfortunately, years of high rainfall, and again lean years, come in long series with great perversity. Rainfall records at Tucson show 6 years, from 1899 to 1904, with less than 10 in. of rain each year, followed by 5 good years with more than 10 in. each year, since which time there have been 4 years with rainfall below the average. The discharge from Sabino Canyon has varied from 2 900 to 46 100 acre-ft. per year. Ground-water levels respond more or less quickly to the variations in rainfall, and in some localities fluctuate over a wide range. Thus, along a section of Pinal Creek, the water-table has been as high as 12 ft. and as low as 85 ft. from the surface. The recent introduction of pump irrigation along the creek will increase the range of fluctuation.

Investigations of safe yield are most fortunate if made during a period when the rainfall is normal. In any case, the investigations

* *Proceedings*, Am. Soc. C. E., for April, 1914.

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should extend over several years, so as to eliminate the effects of storage from year to year. If data on the position of the water-table at many places can be secured during the year for which the estimates are made, the increase or decrease in the storage supply can be computed, and should appear as one item in the balance sheet.

Ground-water reservoirs of large capacity are better equalizers of the water supply than are surface reservoirs, but those of small capacity may be poor equalizers. In nearly all cases the highest utilization of ground-water requires the recognition of the principle of fluctuating area under cultivation. Farmers are loath to admit the necessity for fluctuating area, the principle is not ideal, but in time it will be adopted by Courts and otherwise.

The author's summary of residual losses applies to many valleys perfectly. The outlet loss is not a loss to the State, however, inasmuch as it becomes available in the next valley region. What is termed the artesian loss is, in many cases, not truly artesian in character. This loss and the springs' loss are preventable by pumping operations. The loss by transpiration through trees is preventable, also, inasmuch as the feasible pumping lift is much greater than the limit of penetration of tree roots. Developments of the last 3 years in pumping machinery have doubled the economical limit for depth of pumping. Defining the efficiency of ground-water reservoirs as the ratio of the quantity recovered for irrigation to the total recharge, it is fair to anticipate, and to design works for, an exceedingly high efficiency, higher than many surface reservoirs, which lose from 3 to 10% through evaporation and an equal quantity through losses from a long supply canal.

Methods of increasing the safe yield require some variation from those proposed, in order to make them applicable to conditions in very arid valleys. Efforts to bring about greater absorption of flood-waters are not needed. The floods are wholly absorbed now, except for an occasional season of high rainfall, occurring perhaps once in 15 years. The efficiency of absorption through stream beds must be higher than that of absorption on an exposed slope. The only promising method is the elimination of transpiration through the denudation of the bottom-lands. The native forests and scattered trees should be removed, and, so far as the water supply permits, replaced by vegetation that is useful to man.

In conclusion, the investigations in Owens Valley are timely, for the extensive development of ground-waters in the Southwest points already to the over-draft, and in some places to the exhaustion, of the supply within a few years. The Engineering Profession, and not the Courts of Law, should take the prominent part in the final adjustment of pumping operations to the limiting physical conditions.

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PAPERS AND DISCUSSIONS

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CINDER CONCRETE FLOORS.

Discussion.*

By Messrs. A. W. Buel, Charles C. Hurlbut, W. B. Claflin, F. W. Skinner, Oscar Lowinson, George E. Strehan, and A. L. A. Himmelwright.

Mr. Buel.

A. W. Buel, M. Am. Soc. C. E.—When safe working stresses for cinder concrete have been correctly determined for various specified conditions encountered in practice, it will without much doubt become clearly apparent that there is little, if any, economy in its use for structural elements subject to stress, notwithstanding its lower weight per unit of volume as compared with stone concrete.

The speaker regrets that the author has not taken up the question of the corrosion of embedded steel, as it is of vital importance in a permanent structure, especially as the records of some experiments and considerable experience have shown bad results.

Although a satisfactory series of experiments from which to deduce safe unit stresses for cinder concrete would be a valuable addition to our knowledge of the subject, it would seem advisable first to determine by experiment and experience whether the material is suitable for permanent structures, and that the reinforcing metal is reasonably safe from corrosion. Until this is done, it will hardly be safe to specify cinder concrete for reinforced members or to adopt it as an acceptable material of engineering or of construction—except as a filler—no matter how satisfactory the experimental determinations of its strength may be. In other words, it seems to the speaker that the first experimental and research work with reinforced cinder concrete should be confined to the question of the corrosion of the reinforcing metal, and unless the results are satisfactory on this point it would be a waste of time and money to enter on elaborate experiments to determine proper unit stresses.

* Continued from May, 1914, *Proceedings*.

Mr. Buel. It has been suggested that corrosion sometimes takes place with steel embedded in stone concrete. This is quite true, but as far as the speaker has been able to ascertain corrosion of reinforcing metal in good stone concrete is almost invariably due to one of two causes: electrolysis or palpable defects in design or construction, for both of which there are practical protective measures affording reasonable security. Certainly, one can hardly hope for absolute security, but there is evidence that iron embedded in stone concrete has been preserved rust free for periods as great as 400 years.

The following notes relate to the corrosion of reinforcing metal embedded in cinder concrete: Mr. William H. Fox* states that the results of experience and experiment show serious corrosion in almost all cases, and recommends that with cinder concrete no mixture less rich than 1:1:3, or one still richer in cement, should be used. In a report of a committee read before the Structural Association of San Francisco, it was stated that metal encased in cinder concrete had been corroded, and the committee advised a revision of the code, to exclude the use of cinder concrete. Mr. Fox reports a set of experiments at the Thayer School, with the following conclusion:

"With but one exception, one or more of the three steel pieces in each specimen showed unmistakable signs of corrosion. Apparently it made no difference how the concrete was mixed—wet or dry, tamped or untamped; whether the steam or water treatment was used, the result was the same—rust streaks and spots were found, the difference in the amount of corrosion being imperceptible."

These records seem to make a case against cinders for reinforced concrete that will require something more than explanation, and a good deal of substantial evidence to the contrary, before conservative engineers and architects can safely adopt this material. The advocates of cinder concrete should accept the task of making the necessary demonstrations.

Mr. Hurlbut. CHARLES C. HURLBUT, ASSOC. M. AM. SOC. C. E.—This paper takes a long step toward putting cinder concrete on a rational basis, like other materials of engineering. That it is a perfectly reliable material for many purposes, when made with a good grade of cinders, the speaker is convinced from his own experience. It is not suitable for any work requiring density, such as foundations or outer walls, neither is it suitable for reinforced beams and girders where a stronger material is required; but for floor-slabs between steel beams, as in modern skeleton construction, with relatively short spans and moderate loads, it is very satisfactory. It is not because a great deal of cheap and shoddy work has been done with it, but in spite of that fact, that 90% of the fire-proof floor construction in New York is of cinder concrete. The speaker has repeatedly seen new cinder concrete

* *Engineering News*, May 23d, 1907.

floors in buildings under construction carry loads enormously in excess of those for which they were designed or would ever be called on to carry in actual use. Any one having much experience with cinder concrete in floor-slabs is sure to acquire considerable respect for its load-bearing qualities. Of course, a slab of stone concrete of the same thickness is much stronger, but if the slab is stronger than necessary, there is obviously no gain in increasing the strength, and usually it would not be advisable to decrease the thickness much, even if stone were used. Mr. Hurlbut.

On the other hand, there is no doubt that there has been a large quantity of very inferior cinder concrete work. Sometimes, this is the result of direct "skinning" of the mix, and sometimes it is due to the use of unsuitable cinders. Good cinders are sometimes hard to obtain, even with the best intentions, and on many occasions, the speaker has rejected cinders on the job, which otherwise would have been used in the floor construction. It may be noted in passing that the floor construction in question had been duly "passed" by the Building Department, but it is safe to say that the test on which the approval was based was made with very different cinders from those rejected, illustrating Mr. Waite's objection that the Building Department tests have small relation to what may be expected of any particular floor. The cinders rejected were, in some cases, full of unburned coal and, in other cases, little but fine ashes. Wherever cinders are used, they should be subject to rigid inspection. In the matter of unburned coal in cinders, although it is the speaker's practice not to accept cinders containing more than the small necessary proportion, it is his opinion that the danger from unburned coal in concrete is over-estimated. This opinion is based on a rather crude test made on a partition built of 2-in. cinder concrete blocks. The blocks were literally filled with unburned coal. A small covered enclosure was built and subjected for more than an hour to a very hot wood fire. The coal on the surface was reduced to cinder, but all below the surface seemed to be entirely unaffected.

The objection has been raised, in designing cinder concrete by formula instead of by test, that the material is too variable. This argument does not seem to be well founded. Given a certain system of reinforcement, the strength of any floor-slab will vary from that developed by a full-sized test sample in just the same degree that the materials in it vary from those in the slab tested. It is just as proper, therefore, to designate a safe unit compressive strength and other properties for cinder concrete as it is to assume a safe slab strength based on a test, and far more rational. As a matter of fact, it is a far safer method, for the average strength of cinder concrete can be determined with some approximation to the truth by taking a large number of tests on varying grades of cinders. On the other hand, a

Mr. Hurlbut.

test made on a slab proves little beyond the slab tested. Only one test is made on each "system", and it is safe to say that the cinders entering into these tests are the best that can be obtained. The speaker has witnessed a number of these tests; he has seen some of the test slabs built, and knows that the cinders were better than the average used throughout the city. The charge is not made that any of the tests have been "faked", or even that the materials used were not such as can be obtained in the open market, but merely that the tests of which the speaker has knowledge were made with rather exceptionally clean cinders, which are not always used in actual construction. No proof as to the accuracy of an engineering formula would be deemed conclusive if based on a single test, and yet a single test with an admittedly variable material is here used as a pattern to be followed without variation. It would be possible to define a standard cinder aggregate, the properties of which would not vary beyond reasonable limits, no more, for instance, than timber varies, or some other materials the use of which is based on rational formulas. The proportion of fine ash can readily be determined with a sieve of some standard mesh, and the percentage of unburned material can be determined with sufficient accuracy by means available almost anywhere. These are the two principal uncertain and variable elements in cinder as used in concrete.

A serious objection that has been raised to the use of cinders is their alleged tendency to corrode metal with which they come in contact, and it is to be regretted that the author did not take up this phase of the question. It has been the speaker's opinion that a cinder concrete, if mixed not leaner than 1:2½:5 and placed rather wet, would not injure pipes or steel reinforcement embedded in it. No cases of corrosion clearly attributable to the cinders have come to his attention under these conditions. There are many cases, however, where pipes, embedded in weak cinder fill, have been destroyed, and the presumption has been that the cinders were to blame. It is the speaker's practice to specify and enforce rigidly the requirement that all pipes embedded in the floor construction or fill, where cinders are involved, are to be solidly encased in cement mortar made with 1 part of cement to 2 parts of sand. The Lackawanna Railroad has experienced considerable difficulty with the corrosion of pipes in its stations which, in some cases at least, the engineers have attributed to the effects of contact with cinder concrete. Owing to this belief, they are now prohibiting cinder concrete in every form. In this case, at least some of the trouble was subsequently found to be the result of electrolysis. How much of the corrosion, if any, was actually due to cinders, it would be hard to say. It is quite possible that in other cases of damage to pipes, cinder concrete has been made to bear the blame which should have been attributed to stray currents of electricity.

W. B. CLAFLIN, M. AM. SOC. C. E.—The use of reinforced cinder concrete for floor-slabs and as a fire resistant has reached such proportions, especially in Eastern cities, that its importance as a structural material should be brought to the attention of the Profession at large, and Mr. Waite's paper will be instrumental in accomplishing this. Mr. Claflin.

Some time prior to the introduction of cinder concrete in New York City, architects had become more or less dissatisfied with the materials and methods of fire-proofing then in common use. They were either cumbersome and heavy, or proved inefficient when subjected to great heat. The economic conditions which caused the development of tall buildings and the necessity for reducing the dead loads for the purpose of eliminating the excess of steel in the frames, made it desirable to obtain a material that was durable, fire-resisting, and light. About 16 or 17 years ago, cinder concrete, having been officially tested and approved by the New York Building Department, was introduced. That it has been a success in its limited field of application, time has shown conclusively. The method of testing originally devised, and still followed, does not give accurate results, but was the best that knowledge then available afforded.

For several years after its introduction, the cinder concrete industry was dominated by reputable individuals and companies of financial responsibility who generally controlled a patented method of reinforcing which was not available to others in the trade. It was their custom, as a rule, to arrange for a steady supply of cinders from fairly reliable sources.

The patents referred to cover the forms of reinforcing bars and rods, either plain or deformed, with spacers, clips, and hangers for insuring the proper placing of the reinforcement and holding the concrete protecting the bottom flanges of the steel floor-beams and girders. There is also a rectangular wire mesh with welded intersections, which is one of the best types of reinforcement.

Each concern conducted a series of tests under the direction of the Building Department, and obtained approvals on forms of construction which gave the architects a range of choice sufficiently wide to cover generally all the requirements of practice. The conditions in the trade were fairly stable, with reasonable competition, a good grade of cinders, and responsible concerns to deal with.

Five or six years ago a subsidiary of the Steel Corporation began the manufacture of a triangular-mesh, wire reinforcement, which, having been tested and approved in slabs of various spans and for various quantities of steel, was placed on the market. This is sold to any one having the means to pay for it, and is so cheap that, in many instances, as a result of the ensuing sharp competition, the proprietors are obliged to use it instead of their own methods of rein-

Mr. Claflin.

forcement. No criticism of this form of reinforcement is here implied, for it is one of the best in use to-day.

The trade is not now in such a satisfactory condition as formerly, for the reason that an irresponsible, unskilled, and, too frequently, careless element has appeared, which, through ignorance or willingness to assume risks, underbids its more conservative and responsible competitors. It is practically impossible for the architect to induce the average owner (especially the speculator) to discriminate between the two elements in the trade. Cinder concrete is cinder concrete to him, and it is difficult for him to see beyond the dollars apparently saved by the acceptance of an unduly low estimate.

In times of normal building activity, it is difficult and sometimes impossible for the new element in the trade to obtain cinders of good quality; consequently, they are often taken from household refuse at the dumps, and contain all sorts of foreign matter adversely affecting their quality and the strength of the concrete.

The foregoing has been written for the purpose of showing clearly the influences which have resulted in conditions as they are to-day.

For some time the method of arbitrary tests imposed by the Bureau of Buildings has caused dissatisfaction, not only in the trade, but among architects, because they require much time and are expensive, and because the approval is limited to the exact form tested, no variation whatever being allowed. This means that a firm desiring to use a number of forms of construction must take the time for, and incur the expense of, a test on each one, no matter how slight the difference may be.

This condition has naturally led to the desire, on the part of the cinder concrete interests, for a more rational and simple method of determining the strength of slabs, and the proposal to calculate their stability on the basis of unit stresses is the result. This proposal is logical and highly desirable, if it can be done with safety, but there are points involved which would seem to make it unwise.

Until recently all slabs were tested under uniformly distributed loads, developing unbelievable carrying capacities. The only way to account for this is that the load arches itself, a large and indeterminate portion being transferred to the haunches. Reference to Fig. 2* will show at a glance that arching occurs. In fact, it would require great skill in placing the load to prevent it. We are in the dark, therefore, as to the true strength of the slabs we have used. Within the last two or three years slabs have been tested under concentrated loads at the middle.

As has been intimated, it is difficult to obtain cinders of anywhere near a standard quality. In one building, where the cinder concrete would not set, investigation showed that, although apparently clean,

* *Proceedings*, Am. Soc. C. E., for April, 1914.

on being wet, strong fumes of ammonia gas were liberated. Further inquiry showed that the cinders had come from a brewery in which one of the ammonia pipes had burst over the place where they were stored. In another case, it was found that the cinders (in some concrete which would not set) had come from a sugar refinery and were impregnated with molasses. Fortunately, the work was being done by reputable concerns, and the faulty cinders and concrete were removed, care being taken to have them clean thereafter. Mr. Claflin.

Portland cement is now a standard material, and can hardly be considered an element of uncertainty except in cases where it is not properly protected prior to use, or is mixed in improper proportions. This may be generally applied to the sand used in the city.

The proper inspection of cinder concrete requires a superintendent continually on the work to see that the materials are of the proper kind, and that they are mixed and placed as they should be. It is practically impossible to persuade an owner to incur the expense of a clerk of the works, especially in commercial work. This is another valid reason for a strict law safe-guarding the use of cinders.

The most uncertain of all the elements entering into the composition of any concrete, but more especially cinder concrete, is the so-called human element. Carelessness in or indifference to any one of the many precautions which must be observed in the mixing and placing of concrete may lead to failure.

If unit stresses are used in the calculation of cinder concrete slabs, they should be low enough to discount faulty cinders, carelessness, possible dishonesty, indifference, and lack of inspection. To accomplish this, the unit stresses must be so low that the cinder concrete interests themselves would object on the ground that the quantity of material required would be increased to such an extent that they could not compete successfully with other forms of floor construction. If unit stresses are used, the cinders should be standardized; the law should require that they be washed and screened.

The fire-resisting qualities of cinder concrete have been developed so many times in tests and during actual fires that it would seem that the time has come when they may be accepted as proved and fire tests eliminated in the future.

Recommendations.—The speaker makes the following recommendations:

1. That the Bureau of Buildings conduct a series of tests, approximating as closely as possible commercial conditions in the mixing and placing.

2. That the slabs be tested under concentrated loads at the middle of the spans, with various combinations of span lengths, thicknesses of slab, and areas of reinforcement.

Mr. Claflin.

3. That the results of these tests be divided by a conservative factor of safety and the remainders be tabulated for use as the basis of slab design. For instance, if it is desired to use a slab 4 in. thick, with a 6-ft. span, carrying an applied load of 150 lb. per sq. ft., a given area of reinforcing steel per foot of width should be required.

4. That a ruling be made requiring the washing and screening of all cinders used in all future buildings.

5. That all existing approvals be revoked, after the results of the new tests are available.

The adoption of these recommendations would place the use of cinder concrete on a rational basis and enable those interested to proportion the slabs with a degree of freedom and safety hitherto unknown.

The corrosion of steel embedded in cinder concrete has been mentioned. Several years ago A. N. Talbot, M. Am. Soc. C. E., embedded steel in cinder concrete and concluded that the corrosion was negligible, because, with the wet mixtures used, a film of cement was deposited around the steel which protected it against the action of any chemicals in the cinders tending to cause corrosion.

It is not safe to place unprotected pipes in the cinder fill over slabs. This can probably be explained by the fact that the mixture is usually 1:7 or more, and is placed quite dry, the consequence being that there is not sufficient water present to carry in suspension enough cement to coat the pipes. Piping thus placed should always be given a heavy coat of asphaltic or similar acid-resisting paint.

About the only knowledge we have of the effect of cinder concrete on reinforcing steel in actual construction is derived from the Pabst Building, which was removed (to make way for the Times Building) after having been occupied for 7 years. Corrosion of the steel was found in a few places, but not to a degree affecting its strength. Pipes embedded in the cinder fill were badly corroded.

Mr. Skinner.

F. W. Skinner, M. Am. Soc. C. E.—The speaker's recollection is that, when cinder concrete was first used, it was very largely promoted and intended chiefly as a filler, a light cushion, or a cover for other construction, and it was not expected that it should be subjected to very heavy stresses or that great responsibility should be placed on it. It was found to be convenient, light, and cheap, but it has had larger and larger duties placed on it ever since.

As regards construction, however, concrete of the best quality is one of the most important engineering materials, and one which probably more than any other, varies and is subject to very great modifications through its components and through the manner of its manipulations; it cannot be made and inspected too carefully. Stone concrete without reinforcement is very different from cinder concrete, which is lighter and of more uncertain proportions, and is, and should be, chiefly adopted for mass and fill and for reduction of weight.

Mr. Skinner.

It seems to the speaker that any discussion which gives cinder concrete equal technical weight with a carefully prepared stone concrete, which can be subjected to thorough tests and be made a permanent material of construction, is hardly fair. It is carrying the question beyond the limit of technical considerations, even though it is not beyond the manner in which it is used. In this discussion the most important consideration in regard to cinder concrete is whether it is used to-day in the proper manner and for the proper purposes.

Mr. Lowinson.

OSCAR LOWINSON, ASSOC. M. AM. SOC. C. E.—As an authority on building codes and also on the fire-proofing of buildings, Mr. Waite has no superior. His experience in concrete construction and in the formulation of requirements in building-code matters, is such that his argument—that definite standards for reinforced cinder concrete construction should be established on a mathematical or scientific basis, rather than as a result of tests—entitles him to a respectful hearing.

One great problem in New York City, however, has been to try to make laws which will meet the conditions in the trade; and though this discussion as to the strength of a cinder concrete arch is all right when one knows what material enters into the structure, the trouble has been that frequently the material used is not cinder concrete as an engineer understands the term.

This unfortunate condition of affairs has arisen from the fact that the majority of the concerns who are engaged in cinder concrete construction are utterly irresponsible, have no engineers in their employ, and are subject to no supervision other than that casually given by the building inspector, who usually has only a few moments in the day to devote to the building. Consequently, those familiar with the conditions marvel at the few accidents that occur in New York City, for it is well known that, if it were possible to determine thoroughly the strains in the material which is called concrete, in a large number of cases, it would be found to be strained far beyond the breaking value due solely to the utter disregard of the requirements for reinforced cinder concrete construction.

The expedient was adopted of having tests and giving permits for each type of construction, and for a while this method was effective in that it compelled the installation to correspond to an approved test. Later, however, it was found that the concerns who complied with the tests could no longer compete in construction with the contractors who were willing to install, and the work has gradually drifted so that the concerns who make the tests, to-day, only sell the reinforcement, and the construction is done by other parties.

Under such conditions it is impossible to adopt any kind of a standard, either so-called scientific or practical, of any value; and it is doubtful whether it would be an improvement to change the method of requiring a standard. This certainly is a hardship to a conscien-

Mr. Lowinson.

tious contractor. It is only one of the unfortunate things in the building trade.

This subject is not new, as discussion on the formulation of data with reference to reinforcement has been going on since 1907. New York City has had five proposed building codes, and another is now under way. It is hoped that a building code will be established which will eliminate standards of this type entirely, leaving their determination to the formulation of regulations by the authorities. Possibly, by such a method, a building code may be adopted, which will be based, more or less, on rational ideas.

Heretofore, the difficulty has been the utter irresponsibility of those who are putting in the material and the lack of inspection of the material going into the structure. As a result of this experience it was determined years ago, as Mr. Waite has stated, that it was better to depend on tests with a material of this sort, and establish standards from the basis of such tests, than to attempt to establish standards or unit stresses in a combination material, like reinforced cinder concrete, and the authorities have been very slow in showing any sympathy with the idea that they can establish standards with reference to this mixture.

With a stone mixture one can establish a standard and can generally feel satisfied that the results will be fairly in accord with it. The aggregate is dense, and there is no possibility that it will disintegrate if the slightest inspection is undertaken. With cinders, however, one does not know what to expect; and it is on that account, more than anything else, that the standards in the code of 1912 were established after considerable discussion as to whether or not unit stresses should be permitted in the reinforcement of cinder concrete, and it would have been possible under a similar code, revised as a result of the discussion that took place after the publication of the original code, to establish certain standards of material made on a basis more or less scientific.

The speaker does not think it advisable to establish standards of that sort in any form, other than with reference to the thickness of the material and the quantity of reinforcement per unit or per square foot, by establishing a minimum, because of the fact that with such materials and with mechanics of the type engaged in that work in New York City, to-day, it is not advisable to go further.

With reference to the corrosion of pipes when buried in concrete, experience has taught us that the only way to feel sure that pipes will last is to insulate them, and thereby prevent the corrosion from getting at them. A wrought-iron steam pipe, embedded in a concrete floor, is good for from 2 to 5 years. It is the custom, especially in cellars, to try to keep the return pipe above the floor level. Corrosion in pipes is practically due to oxidation, caused by the sulphur in

the cinders, or by the porosity of the concrete permitting the moisture, which is alkaline, to attack the pipe. Mr. Lowinson.

There has been a great deal of trouble with hot-water supply pipes, when buried; they corrode more rapidly than the cold-water pipes. Frequently, over these pipes there is a layer of plaster of Paris, or cement from a tile floor, and the corrosion is generally attributed to the above-mentioned cause, to which is added expansion and contraction of the pipe caused by variations in temperature.

It has been found that corrosion occurs on the outside and also on the inside of the pipe. Chemists may be able to explain why the corrosion takes place on the inside as well as on the outside, and why it is more active in hot-water than in cold-water pipes.

GEORGE E. STREHAN, JUN. AM. SOC. C. E. (by letter).—The author's object in recommending that the application of cinder concrete in fire-proof floor construction be placed on a so-called scientific basis is to be commended. However, the writer fails to find any plausible basis in the paper on which to establish constants for calculation and unit stresses for design. The author states that there are sufficient precedents for establishing such factors, but a careful perusal of the paper does not elicit much information of great value along this line. Table 3, showing the physical and elastic characteristics of cinder concrete, which the author presents, was taken from a preliminary report of an investigation being made at Columbia University, and these data in themselves are not sufficiently comprehensive to be used as a basis for design. Mr. Strehan.

Cinder concrete, as applied in New York City and vicinity in fire-proof floor filling between steel beams, is a material with which the author should be thoroughly familiar from long years of experience, both as a contractor and as an "expert tester." The knowledge thus gained should have convinced him that such a floor, built between rigid I-beam supports on ordinary spans of from 6 to 8 ft., is an entirely different structure from reinforced concrete of the slab, beam, and girder type. On spans up to 8 ft., and with a thickness of not less than one-twentieth of the slab length, the writer's investigations have shown that such factors as arch action, tension in the concrete, and the trussing effect of continuous reinforcement, all of which the author neglects, are important features in floor construction of this type.

The author ridicules the term "continuity" thus used without apparently realizing that it is not the continuity of the concrete slab which is referred to, but the anchoring of the reinforcement over the steel beam, and although he has little faith in it, he still feels that such continuity obtaining in the test construction and not in actual buildings has resulted in the excessive load approvals. Based on both theory and test, it can be shown that this strength, of the worst condition that occurs in building construction, that is, the

Mr. Strehan.

end slab adjoining an open bay or exterior wall, is about three-fifths of that of the test construction as ordinarily installed. The author failed to note that the tests conducted by the Bureau of Buildings, with the resulting approvals to which he refers, were made on the basis of a factor of safety of 10. This, in the end-panel condition of actual practice, would correspond to a factor of safety of 6 for the approved loads.

The writer does not undertake to defend the method of handling cinder concrete floor construction which obtains in New York City, but believes that a rational basis for design should be established. With this end in view, a paper is now in course of preparation which it is hoped will soon be ready for publication. The investigation has included a study of the preparation of the raw materials, with the object of improving the mixture. Very little, if any, benefit has been derived by screening the cinders; in fact, the resulting concrete has shown a decrease in strength when compared with the ordinary 1:2:5 mixture in which hard coal cinders, as received from the boiler, are used.

The author states that the form of reinforcement is immaterial. Exception must be taken to this statement, because, in some tests of slabs reinforced with plain and deformed rods, the plain squares and rounds failed by the slipping of the reinforcement. The question of adhesion must undoubtedly be taken into account in the design. The writer wishes to emphasize further the fact that if the reinforcement is anchored to the steel girders, the resulting construction differs radically from the type of slab, simply reinforced, on which the author bases his plea for design. The extension of cinder concrete to general application in slabs, beams, and girders should in no case be recommended, in view of the variableness of the material. Its use between steel beams as a special application has undoubtedly proved satisfactory; but, even in this case, there should be greater caution than is now generally shown in the selection and preparation of the raw materials for the concrete.

Mr. Himmelwright.

A. L. A. HIMMELWRIGHT, M. AM. SOC. C. E. (by letter).—The subject of cinder concrete floors is one in which the writer is greatly interested. The idea of rationalizing the theory and practice of their construction is not new, and would be most desirable if a practical and correct formula could be evolved that would be generally applicable to the different forms of reinforcement and methods of using it. The writer spent much time years ago in attempting to develop such a formula, and was obliged to give up the attempt.

Mr. Waite's arguments are relevant, but, unfortunately, they are applicable only to the ordinary and usual types and methods of reinforcement, and would not include numerous others equally desirable.

Mr. Himmelwright.

Take, for example, an abnormal case, such as a reinforcing metal without rigid intersections, of which hexagonal mesh netting is an extreme case. In this netting the metal does not actually get into tension until the concrete slab cracks and fails, when it acts as a suspender. This material, of course, is unsuitable for the purpose of reinforcement, but it illustrates a type, such as "crimped" reinforcement, which is actually sometimes used. Then, again, there are forms of bar reinforcement, such as what is known as the Roebling System B, with 2 by $\frac{3}{16}$-in. flat bars set on edge. The reinforcing bars in that position in the slab have considerable value as beams, and the actual strength cannot be even approximated to by any formula that gives fair results with a bar of square or round section.

This type of construction is one of the most fire-resisting flat-slab methods now in use, because the bars, extending deep into the concrete, are protected by it, and in that position do not become heated as quickly as when the center of section of the reinforcing metal is nearer the under surface. When any of the ordinary formulas are applied to this type of flooring, they scarcely give it strength enough to support its own weight, whereas actual tests have shown its strength to equal, and in some cases exceed, that of the average of equivalent square and round bar reinforcement used in the ordinary manner.

The great difficulty in evolving a rational theory of design is that the strength is not the only important feature of fire-proof construction; fire resistance and durability are even more important. Conditions which will produce the greatest strength generally result in the poorest fire resistance; that is to say, the farther away the reinforcing metal is from the neutral axis and the nearer it approaches the underside of the slab, the greater will be the strength; on the other hand, the nearer the reinforcing metal approaches the under side of the slab, the thinner will be the protective covering and the more quickly the reinforcing metal will be heated in a fire so as to become weak.

The question of durability is most important. Exposure to the elements, which is a necessary condition in practical building construction, always causes initial rusting. When the reinforcing metal is in the form of bars or rods of considerable size and section, the rusting is not important, and does not materially weaken the metal. When, however, the reinforcement is in the form of wire or small mesh made from light sheet-metal or similar material, the rusting affects the strength very materially by reducing the effective section, and when embedded in wet concrete it continues to rust in rainy weather and until the building is roofed over. In extreme cases such reinforcing metal under normal conditions becomes weakened by oxidation, so that it retains only a fraction of the strength shown by the formula.

Mr. Himmelwright.

Density is another factor that has large importance in cinder concrete fire-proofing. When stone aggregates are used and the concrete is thoroughly rammed so as to eliminate all voids, the action of the concrete approaches more nearly to that of the solid stone of which the aggregates are composed, and it cracks much more readily when exposed to heat and sudden cooling than when the concrete is full of voids. This, in fact, is the principal reason that cinder concrete develops higher fire resistance than any concrete with stone aggregates. It is impossible to eliminate all the voids in the cinder aggregate by ramming, as such voids are always present in the aggregates themselves, to a greater or less extent, even if the cementing material is free from them.

Tests made by the writer conclusively demonstrated that the more porous the concrete the less likely it is to crack and fail under sudden and violent changes of temperature. Cinders crushed so as to pass a ¾-in. screen, made into concrete mixed in the proportion of 1 : 2 : 5, and simply spread over the forms and leveled and patted down with shovels in segmental arch construction, will develop ample strength to carry ordinary loads up to, say, 200 lb., with a safety factor of 5. The material manipulated in this way will be extremely porous, light in weight, and exceed in fire resistance any other material known to the writer, except possibly a concrete made from "tetzlonti," a porous lava rock which the writer has used in a number of buildings in the City of Mexico.

It is well known, of course, that density is desirable, as tending to prevent oxidation of the steel reinforcement, as oxidation takes place more rapidly where voids occur in the surrounding concrete; but this oxidation practically ceases when the concrete becomes perfectly dry after a building is enclosed and finished. This phase of the problem, therefore, is of minor importance, especially when the reinforcing metal is in the preferred form of rods or bars of considerable section.

The economic advantage of voids in securing lightness is much more important than the initial oxidation, but it has never received the attention it deserves. A concrete made and deposited as above described will weigh only from 70 to 75 lb. per cu. ft., as against 95 to 100 lb. when thoroughly rammed. This is a gain of 25% in the dead load, in favor of economy, and, as above stated, ample strength is also obtained.

From what precedes it will be apparent that certain qualities and conditions which result in improved fire resistance, both in the form of the reinforcing metal and in the concrete, tend toward weakness, so that we have two conflicting elements in the design of cinder concrete floors which will forever prohibit the development of a practical rational formula of general application.

Mr. Himmelwright.

An important feature of this whole problem is the fact that it is a mistake to legislate or inaugurate regulations that are restrictive and tend toward the development and improvement of only a few forms of construction. The error of this, perhaps, is more plainly illustrated in the regulations governing theater construction than in anything else. All the building laws at the present time specify refinements of the conventional design for theaters, and absolutely prohibit other designs which might provide greater safety and result in larger seating capacity. It is always better, in preparing building codes and city ordinances, to specify results to be obtained rather than detailed specifications.

Although strength in cinder concrete floors is a most desirable feature, others, equally desirable, should not be needlessly sacrificed and discouraged by ill-advised regulations. To make all forms of construction comply with a rational theory would immediately eliminate some of the best methods, as it would be obviously impossible, with the same materials and at the same cost, to develop a superior fire-resisting construction which would show by formula a strength equal to the ordinary method with the reinforcing metal as near to the under side of the slab as the law would permit.

The writer is disappointed that more progress has not been made in the direction of economy and in improving the fire resistance of cinder concrete floors. Practically all the progress during the last 5 years has been in the direction of securing the maximum strength with any given reinforcement; the protection of the latter being controlled by building codes and regulations, the thickness of the concrete protection varying from $\frac{1}{2}$ to $1\frac{1}{4}$ in. in different cities. The general tendency in this one direction of securing strength, which is largely the result of existing restrictive codes, has no doubt discouraged and minimized improvements in other directions, and has accentuated the supposed desirability of rationalizing all forms of cinder concrete construction.

The best that can be done with the problem is to develop a theory and formula in accordance with the results of the usual and more common forms of cinder concrete floors, as proposed by Mr. Waite, and, in addition, make a separate provision for the determination by test of the strength of those special forms of construction to which the formula and rational theory do not apply. This course has recently been followed in the most up-to-date building codes, in order not to discourage or prohibit methods which aim at superior fire resistance, lightness, and other economies.

There is room for great improvement in the matter of test loading of floors. In the past this has not been done in a scientific manner, and the varying and conflicting results of many of the tests is largely the result of improper or so-called "fake" loading. This is a subject

Mr. Himmelwright.

which in itself can well constitute a separate paper, and is so important that the writer trusts that some one who is well-qualified will give attention to it in the near future.

Another great stumbling block in cinder concrete construction is in the definition of what constitutes a good quality of cinders. There is widespread ignorance in regard to this material, the popular idea being that it is coal ashes from ordinary stoves and furnaces, which contain from 50 to 75% of soluble sulphates and silicates in the form of white powder and comparatively soft whitish lumps.

Those who have this idea of cinders sometimes specify that they shall be screened, discarding all fine material, hoping in this way to eliminate the fine white ash, and, by retaining only the coarser portion, they expect to obtain suitable material. Ordinary coal ashes, if treated in this way, will still contain a large proportion of the soluble sulphates in coarse lumps.

Cinders with any considerable quantity of soluble sulphates are entirely unsuited for concrete aggregates, and should be prohibited. A good quality is obtained from large steam boiler plants in which the cinders are cooled by spraying. They contain practically no soluble sulphates or silicates, and from 95 to 98% of the fine material is grit fully equal to sand of good quality for the purpose of making concrete. When a building law requires screening, it can readily be understood that excellent material (the fine grit) is thus prohibited and wasted, which involves useless expense and a large loss in economy of construction. Locomotive cinders—on account of the coating of soot—and cinders from gas plants, are also unsuited for concrete. The extremely coarse clinker from bituminous coal is objectionable, as is, in fact, any coarse material.

To get uniform and satisfactory results, all cinders should be passed through rollers or crushers which will reduce them to a size that will pass a ¾-in. square-mesh screen. If sand is added to the fine material, so as to make a mixture in the porportion of 1 part of high-grade Portland cement to 2 parts of fine material and sand, and 5 parts of crushed cinder aggregate, a most excellent fire-resisting concrete will result.

A good definition for a suitable quality of cinders in a building code is the following:

"Cinders, where referred to in this code, shall be construed to mean the residue of coal consumed in large steam boiler plants, free from soluble sulphates and silicates, and in which the fine material is composed wholly of grit. (Note:—Ashes or cinders from ordinary coal stoves, ranges, hot-air furnaces, small steam boilers, or locomotive or gas-plant cinders, shall in no case be used for aggregates for cinder concrete.)"

An imaginary difficulty with cinders is the large quantity of unburned coal they are supposed to contain. Some years ago, in the

writer's experience, work on one of the largest buildings in New York City was suddenly stopped by the architect. His inspector happened to be in the basement one day and saw a considerable quantity of black particles in the cinder. Cinders of the same quality had been used on the entire job, which was then half completed. The foreman in charge of the fire-proofing endeavored to satisfy the inspector by explaining that not all the black particles were unburned coal, but the inspector insisted that the work be stopped forthwith, and the architect so ordered. The inspector then selected a quart of the black particles from the cinder pile, took them to his own chemist, and had an analysis made. The result was that only 7% of the particles selected were found actually to be coal, and 93% consisted of stone, slate, and other impurities resembling coal. Although there appeared to be as much as 10% of unburned coal in the cinder supply, this analysis proved that there was in reality less than 1 per cent. Mr. Himmelwright.

The foregoing are a few special features and conditions which the writer has noted in his experience in connection with cinder concrete fire-proofing. He has not attempted to go into the matter thoroughly, or in any detail, as he has not the time or opportunity to do so; but attention is called to these features of the problem in order to indicate that it is larger and more complex than is realized by the average person, especially by those whose experience is limited to a few tests made with the ordinary forms of reinforcement, and who consider only the question of strength.

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PAPERS AND DISCUSSIONS

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REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

Discussion.*

BY MESSRS. J. J. H. MUIRHEAD, W. J. BARNEY, EUGENE W. STERN, R. D. COOMBS, JOEL J. PEMOFF, WILLIAM GOLDSMITH, DEWITT C. WEBB, JOSHUA F. RAMSBOTHAM, AND CHANDLER DAVIS.

Mr. Muirhead.

J. J. H. MUIRHEAD, M. AM. SOC. C. E.—In regard to concrete dock construction, the speaker always looks on Para as one of the examples of what not to do. While there, he calculated that, approximately, £5 000 000, sterling, had been spent for the accommodation of five ships. Of course, that calculation is not altogether fair, yet it is not unfair, because it illustrates a system of finance radically different from that which is customary in the United States. In fact, it is a system that would not be tolerated in American undertakings. It is true that comparatively little more money was required to complete the work, which now accommodates a large number of ships, nevertheless, the initial outlay was enormous, and quite unwarranted by the results.

The speaker regrets that as a Society we spend too much time in the discussion of details and do not study the general schemes as we ought. He knows of no justification for the construction of such expensive port works at Para, beyond the fact that the concessioners and contractors made a profit. True, the civil engineers made something, also, but not by any means enough to compensate for some forty lives lost by yellow fever. Quite a brisk shipping business has

*This discussion (of the paper by Harrison S. Taft, Esq., published in April, 1914, *Proceedings*, and presented at the meeting of May 20th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Muirhead.

been done in materials for the improvement of the City of Belem de Para itself, but that is temporary.

The only trade that Para has, or is ever likely to have, is rubber. Are not our friends, the mechanical engineers, doing all they can to devise substitutes for rubber? It is being planted in many countries. The conditions under which it is obtained in Para and the neighboring States are so bad that it is a very precarious industry. The fluctuations in price are great, taxation is unreasonable, and "ladrones" are only too common.

The chief work at Para consisted in a bulkhead wall, built on the "slice block" principle, extending along one bank of the River Amazon, in front of the city, which provided 35 ft. of water alongside the wall. The speaker cannot see that this system was at all suitable for the conditions to be met on the Amazon. The glaring defect was the absence of stone with which to make concrete. Cement was imported from England and sand of excellent quality was obtained a day's journey down the river. The blocks were made of cement and sand only.

At the same time, clay suitable for brick-making was being dredged immediately in front of the wall and disposed of at sea or in the deep parts of the Amazon quite a distance away. From experience gained by the speaker in Kingston, Jamaica, and from the opinions of men who worked there when that city was being rebuilt after the earthquake, much the finest concrete-making material one can have is broken brick. On one occasion, the speaker had to remove some broken-brick concrete, and although it had set only a comparatively short time, the difficulty of removal has given him a very great respect for concrete of this class. He has never heard of making brick and deliberately breaking it up when almost new as a substitute for gravel or broken stone. Under given conditions, the experiment should have been made. The design contemplated 30-ton blocks. Had they been made in the proportion of 1 part cement, 2 parts sand, and 4 parts broken brick, a great saving would have been effected over 1 part cement to 2 or even 3 parts sand.

In some places, stone for ballasting was necessary. In these cases, a very fair grade of iron ore was used. This was also a waste of good material. Again, the speaker would suggest broken brick, even when the brick has to be made and then broken by mechanical means.

Mr. Barney.

W. J. Barney, Assoc. M. Am. Soc. C. E.—This paper serves at least one purpose, in that it places data relating to dock construction before the members of the Society. The great problem in such construction is concrete *versus* timber, and Mr. Taft has gathered together original references and made suggestions as to where information thereon may be obtained. The paper represents weeks of compilation in the library and by correspondence, and involves a great deal of hard research

work. The speaker hopes that it will be given the benefit of careful discussion of the many points involved in concrete *versus* timber pier construction and a combination of concrete and timber piers. Mr. Barney.

In Portland, Ore., it was a great question whether concrete or concrete and timber combined should be adopted. The city is in a timber district, yet the necessity of avoiding fire risks seemed to compel the use of concrete, and a combination of timber below the water line with concrete above that was used in order to avoid such risk and, at the same time, to keep down the cost. To the speaker, the great question in such matters has always seemed to be the "commercial life" of the piers; that is a vital factor which is frequently ignored. In foreign ports, pier construction has had practically to be dynamited out, because its "commercial life" had passed away and new construction was necessary for larger vessels and changed conditions.

These questions should be considered and discussed more widely than they have been, particularly in connection with this paper.

EUGENE W. STERN, M. AM. SOC. C. E.—There are some general considerations in this paper which it might be well to review carefully. Mr. Stern.

It will doubtless be conceded that it is desirable in many cases to substitute a more durable material than wood for docks and piers, provided the cost is not prohibitive, and its permanency in sea water can be assured.

Mr. Taft has gone to a great deal of trouble to gather information and present it in concise form, but he treats rather indifferently the fact that in the United States there have been some serious and disastrous experiences with concrete structures in sea water. He cites many cases in which concrete has stood well, but does not dwell sufficiently on the failures.

Some 10 years ago, the speaker's attention was first called to the disintegration of concrete in sea water on some pier work in the lower part of the Hudson River. Since that time he has been following this subject carefully, with the result that he is convinced that there is as yet considerable uncertainty as to the action of concrete in sea water, and until that uncertainty has been removed, engineers must go slowly.

At Atlantic City quite a number of concrete structures are exposed to the action of sea water. A few years ago, the speaker investigated the condition of almost all these structures. It was just about the time when the large cylindrical concrete piles were being sunk for the Boardwalk, but the speaker has not been able since to examine these particular piles. Every structure in Atlantic City that the speaker examined, from one end to the other, which came in contact with sea water, showed more or less deterioration, and it is believed

Mr. Stern.

that any one with open mind, who went there to investigate, would be similarly convinced. The reinforced concrete structures built over the sea showed, not only the effects of frost and the action of waves, but also chemical disintegration.

There are at least three kinds of action which may cause deterioration in concrete structures exposed to sea water:

1.—The effect of the impact of the waves;
2.—The effect of frost;
3.—The effect of chemical action.

At Atlantic City, it seemed to the speaker that a great deal of disintegration was due to chemical action. In places the concrete had cracked so much that the salt water had penetrated to the reinforcement, which had rusted badly. The concrete had also swelled and cracked off in places. In one of the large piers, built a few years ago and projecting into the ocean, the piles had shells of boiler iron, 18 in. in diameter, riveted together. These had been sunk and then filled with concrete. At the time the speaker examined this pier (probably then about 10 years old) nearly every one of the cylinders had been split open by some great internal pressure, due without doubt to chemical action having taken place in the concrete, caused by sea water entering through the joints of the shell, which were not caulked. Many vertical joints in the iron shells were burst open and the rivet heads sheared off. The joints had opened to such an extent that part of the concrete filling could be raked out, and was found to be almost in a powdered condition.

A series of valuable experiments is now being made by the Aberthaw Construction Company, of Boston. About 5 years ago, twenty-four square piles, about 16 by 16 in. and 16 ft. long, were prepared. These were placed in the sea at the Charlestown Navy Yard, their tops being about 18 in. out of water at high tide, and their bottoms about 4½ ft. under water at low tide, the mean tidal range being 10 ft. These piles were examined in December, 1913, after having been in place about 5 years. The published report of the Aberthaw Construction Company shows that only one of these twenty-four piles did not show disintegration of some kind. One pile was eaten through; others were eaten into 4 or 5 in.; others showed slight defects on the edges only.

The one pile which stood the test was made of German iron-ore cement of a rich wet mixture, namely, 1:1:2. Other piles made of the same mixture, but of a native cement, stood well, but showed some disintegration. Now, the question arises, what will the result be after 20, or 30, or 50 years?

These tests prove that the cement chemists and manufacturers must give the engineer a cement that will better withstand sea water.

Mr. Stern.

It is all very well to say that concrete docks should be built. The speaker believes in having permanent dock structures, if they can be built economically, and with reasonable assurance that they will last, but there must be this assurance before such construction is undertaken on an extensive scale.

There is no doubt that something better than wood is often very necessary. On the Pacific Coast, the speaker has seen large piles which have been completely cut through by marine borers in a few months.

Engineers are very anxious to get something better than wood for this purpose, and if German cement makers can produce a cement which, at the end of 5 years, shows no disintegration in sea water, those of America should be able to do as well.

The author brings up the question as to the effects of frost on masonry structures in the Great Lakes. The speaker has observed that in the lock at Sault Ste. Marie, on the Canadian side, the walls built of limestone show, at the water line, disintegration due to frost. The stone seems to be rather porous, and in some places the cavities are several inches in depth. There is no doubt that, wherever concrete is porous, frost action will be more or less harmful, so that it is necessary to get a dense mixture for concrete exposed to frost action, whether in fresh or sea water.

Mr. Coombs.

R. D. Coombs, M. Am. Soc. C. E.—If the speaker remembers the various reports of the Masonry Committee of the American Railway Engineering Association, the gist of all of them has been that, for waterproofing, or preventing decomposition in salt water, the main thing is to make a very dense concrete, and that is practically the summation of all other reports.

Mr. Pemoff.

Joel J. Pemoff, M. Am. Soc. C. E.—The Dock Department of New York City has been building sea walls of concrete blocks set in place under water for the the last 40 years. The cement used in these blocks is such as is usually specified for first-class work. The Department has never met with failure, or even deterioration, in any portion of this work, so far as could be ascertained by submarine examinations made by divers. Only in a very few places can disintegration be noticed, and those places are between high and low water.

The great success of the Dock Department with its submarine work is due to the mixture used, 1:2:5; to the dense surface produced on the blocks; to the fact that Portland cement of the very best quality is always specified and obtained for this work; and probably also to the condition of the waters in this harbor, which carry a great deal of fine matter in suspension, tending to form a slimy coating on the blocks and thereby increasing the imperviousness of the outer skin.

One of the causes of disintegration between high and low water

Mr. Pemoff. is the fact that this portion of the work is usually deposited *in situ* while the tide is low, and the early removal of the mould boards allows the sea water to reach the concrete before it is sufficiently seasoned to withstand its effects. The disintegration of exposed concrete surfaces in New York Harbor is rather the exception than the rule.

Mr. Goldsmith. WILLIAM GOLDSMITH, ASSOC. M. AM. SOC. C. E.—Reinforced concrete piles, as well as a new type of reinforced concrete bridge resting on piles, have been used in many places throughout the Philippine Islands. These piles are usually 16 by 16 in., approximately 40 ft. long, and have a 1½-in. steel pipe running through their centers so that they may be jetted into place. Pantal and Calmay Bridges, being, respectively, 300 and 600 ft. long, are on piles of this type.

There are five piles in a bent, and the bents are 20 ft. from center to center. Reinforced concrete girders rest on these piles, the bridge being designed to support a 12-ton road roller.

These bridges have been in use about 5 years, and many more have been built. They have stood the test of salt water very well, and show no deterioration due to this cause. The cement used in their manufacture is "Green Island cement," made in China, the concrete mixture being 1:2:4.

Wooden piles in the same locations have lasted only 2 or 3 months. The teredo is very active in these waters, a wooden pile bridge being hardly completed when the work of the teredo would be so evident as to make the structure dangerous. It would seem, therefore, that wooden bridges have no place in Philippine waters and that concrete is the only lasting material which can be used for such structures.

The type of bridge referred to is also particularly adapted for Philippine construction on account of its low cost in comparison with that of arch bridges of wood or even of steel, the life of which is also of short duration.

Great care is taken in the testing of Portland cement in the Philippine Islands. The "standard method for analysis of cement, proposed by the Committee on Uniformity in Analysis of Materials for the Portland Cement Industry," is used. In the field, each district engineer has sufficient apparatus to make 24-hour and 7-day tensile tests, thus giving an independent check on the cement inspection.

Mr. Webb. DEWITT C. WEBB, M. AM. SOC. C. E. (by letter).—This paper is a valuable compilation of information in regard to concrete structures in water, particularly in sea water.

Some of the author's statements in regard to the deterioration of concrete in Boston Harbor are not in accordance with the writer's observations. The author, in laying stress on the action of frost in disintegrating concrete, states: "In Boston Harbor 12° below zero is not uncommon."* An examination of the records of the Weather

**Proceedings*, Am. Soc. C. E., April, 1914, p. 1002.

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Bureau for the 41 years, from 1873 to 1913, inclusive, shows only two days during that period when the minimum temperature was 12° below zero or lower. During the same period, there were only 121 days, or an average of less than 3 per year, when the minimum temperature was zero or lower.

The impression conveyed by the author, that frost has been the principal cause of the disintegration of concrete structures in Boston Harbor, appears to coincide with the general opinion. During the past 5 years, the writer has had opportunities for careful examination of many of these structures, and has supervised repairs to several of them. The result of this experience has been the formation of a very firm opinion that although frost and wave action have had some effect, the principal destructive agent has been chemical action. If frost were the most active destructive element, the concrete behind and adjacent to the disintegrated areas would, it is thought, be sound and hard. In several instances this was found not to be the case. On the contrary, the concrete which was cut away to permit a new facing to be applied, was soft, and, so far as could be determined, this condition extended through the entire mass. Pockets and streaks of a white putty-like substance were found in the concrete removed. This material was chemically active to such an extent that it burned the hands of the workmen. Neither the appearance of this substance nor the manner in which it occurred suggested that it might be laitance, nor does its chemical composition, as given below, appear to indicate such an explanation:

	Percentage.
Lime (CaO)	31.11
Carbon dioxide (CO_2)	9.63
Silica (SiO_2)	8.87
Sulphuric anhydride (SO_3)	6.68
Magnesia (MgO)	5.35
Iron and alumina oxides	4.38
Insoluble residue	5.86
Combined water	12.05
Moisture	13.66
Salt and undetermined	2.41
	100.00

The cement used originally in this work was a well-known brand, and the records of the chemical analyses show the following average results:

	Percentage.
SiO_2	22.50
$(Al_2Fe_2)O_3$	9.55
CaO	61.98
MgO	2.80
SO_3	1.33

Mr. Webb.

It appears to the writer that these statements tend to confirm the theory that the chemical action of sea water on concrete is due to the flux and reflux of the salt water in and out of the voids, gradually causing magnesia hydrate to form and abstracting the lime from the cement. It is only when concrete is porous and exposed to the unbalanced or intermittent action of sea water that trouble occurs, as it is the continuous percolation and renewal of the sea water which does the mischief. For this reason concrete below low tide is usually secure.

The remedy is not chemical, but mechanical. The sea water must be prevented from penetrating the concrete. This can best be done by the use of a dense, rich mixture, placed wet. It is desirable, when possible, to permit the concrete to harden before exposing it to sea water, but this is not absolutely essential, as can be proved by concrete structures in Boston Harbor which were not pre-moulded or built inside of coffer-dams.

The damage to the concrete sea wall, at Lynn, Mass., mentioned by the author, is believed to be due almost entirely to the pounding of the waves and wave-borne materials such as driftwood, pebbles, and larger stones from the beach. The repairs by the cement gun, when last observed, did not appear to the writer to be entirely effective. This might be expected, as the repair work was largely near low-water mark, and the rising tide would bring the repaired areas within reach of wave action shortly after the cement-gun plaster was applied.

The writer believes that concrete, properly proportioned, mixed, and placed, is an entirely suitable material for structures in sea water in any climate. It is desirable, however, to provide a thin facing of stone, hard brick, or similar material between high and low water, where it is exposed to destructive action from waves and wave-borne objects.

Mr. Ramsbotham.

JOSHUA F. RAMSBOTHAM, M. AM. SOC. C. E. (by letter).—Mr. Taft is to be congratulated on the clear and concise conclusions which he has drawn from his evidently deep study of the experiences of all nations in the use of ferro-concrete as applied to marine works.

The writer agrees with the conclusions drawn, and would like to observe that, as far as he is aware, there is no other paper or book which covers, to the same extent, the large range under review.

In regard to the use of cement under sea water: It was the writer's privilege to remove 68 404 cu. yd. of concrete, made in the earliest days of Portland cement concrete, in the Port of Liverpool, while acting as Resident Engineer on the Brunswick Dock Extension. The concrete, which was almost 40 years old, was perfect, and in no single instance was there any sign of decay or shoddy work. During that period of 40 years, the work had been under sea water, the walls, etc., in the first instance having been built in the dry.

Mr. Ramsbotham.

It is the writer's firm conviction that in building graving docks, walls, etc., it is unwise to place freshly mixed concrete in water, for the reasons given by Mr. Taft.

It is doubtful whether true reasoning has been observed as regards the Cattle Wharf at Liverpool, the deck being a considerable height above the water, and the writer cannot help thinking that "bloom" on the steel may have been the cause necessitating repairs. The author has not mentioned this important factor in the problem, but it is an absolute necessity that all bloom, dirt, or rust, should be removed before concrete is placed in the forms; if a skin of bloom is on the steel, the cement does not reach the metal, and trouble may be expected in the future.

The writer does not agree with the four conclusions drawn by an English engineer, as ferro-concrete work is certainly not indestructible, either from fire or shock, and, further, it is by no means easy to repair, in fact one can hardly conceive a more difficult job than the repair of a partly destroyed ferro-concrete structure; it would resolve itself into points, wedges, and hack-saws, and under water it would surely be most difficult.

Ferro-concrete has its proper sphere of usefulness as applied to marine works, but, at the same time, and surely to a large extent, the question is resolved by prime cost and permanency. At Liverpool, ferro-concrete was approached by Mr. Anthony G. Lyster, then Engineer-in-Chief to the Mersey Docks and Harbour Board, with care and caution, and all the work was done in such a manner that it was under review, and under no extreme circumstances could it interfere with the traffic of the Port. The policy was to make M. Hennebique not only responsible for the design, but also solely responsible for carrying out the work in a suitable manner. At the same time, Mr. Lyster had his own independent staff, who supervised and checked everything.

The writer maintains that the design cannot be separated from the execution, and that a dual control results in extravagance and inefficiency, possibly accompanied by disaster.

Mr. Davis.

CHANDLER DAVIS, M. AM. SOC. C. E. (by letter).—This paper, covering, as it does, the most important structures of the world, is most interesting and valuable, and deserves the careful study of engineers who are interested in this subject.

The most vulnerable part of maritime structures is that which lies below high water, as the materials are exposed to the action of water, which, as a rule, is not over clean. Most harbors are near or in large cities which generally empty their sewers into the rivers or bays on which they are located, and thus docks and other structures are exposed to the action of, not only sewage, but also of refuse discharged by factories, chemical works, and similar plants. These

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local conditions must be taken into consideration in drawing the specifications under which the materials are furnished and inspected.

The laboratory testing of the materials is as important as the field inspection, and should not be neglected. It is not sufficient to see that the plans are properly carried out, but every effort should be made to see that the supplies conform to carefully drawn specifications. These should read so as to cover every test to insure the use of only first-class materials, suitable for the work in view, and acceptable to the engineer. The contractor should be obliged to have sufficient material on hand to permit of a thorough test and inspection before it is used in the work. This applies especially to cement, and none should be permitted to be used under the 24-hour test. In fact, if it is possible, the cement should be held 28 days, or even longer, in order to determine its increase in strength over the 7-day test. This is important if the concrete is to be used in water, and especially if an impervious wall is required, such as would be the case in the construction of a dry dock.

Many specifications state the component parts of which the materials should be made. This, in certain instances, can be done, but, on the whole, it is safer to limit the quantities of those ingredients which, necessarily, must be a part of the finished substance, but are to an extent injurious, and leave the rest to the manufacturer. By using only well-known and thoroughly tested brands, in which the injurious components have been reduced to a practical minimum, the engineer obtains the quality and durability for which he is looking. The fact that known and tried supplies are being furnished does not relieve the engineer, however, of his duty as an inspector, and he must still insist on clean, pure materials, free from all foreign matter and substances which are likely to injure the material and impair its strength and durability, and that they should be free from adulterants, which, perhaps are harmless in themselves, but have no special value.

The engineer should insist on the use of the best and only first-class materials, and should see that all analyses and tests are carefully made by competent assistants, reserving for himself alone the right to accept materials, which, although they do not quite conform to the specifications, may, in his opinion, be satisfactory.

Reinforced concrete has been used extensively throughout the world, and with great success. There have been failures, but the general results have been good and satisfactory.

The use of concrete and reinforced concrete in dock work is increasing annually. It has been applied to quay walls, storage and cold storage warehouses, piers, and piles. Great care, however, must be used in making, manipulating, and placing the concrete, for, if used in salt water, it must be made impervious; this also applies to concrete placed in fresh water. In order to obtain impervious concrete,

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the percentage of mortar in the mixture should be larger than that of the voids in the stone, and the quantity of cement in the mortar should never be less than one-half of the quantity of sand. If this is done, the mortar will fill all the voids in the stone and properly cover it with mortar, the concrete will then set properly, the mass will be thoroughly cemented together, and the structure will be impervious to water. In certain instances the face of the work may be protected by a layer of richer concrete. Assuming a quay wall to be built of 1:2:5 concrete, the proportions in the outer 18 to 24 in. which is exposed to the action of the water, would be 1:2:3. This richer concrete will resist the flow of water through it, and the weight of the whole wall will carry the superimposed loads and resist the pressure due to the filling behind it. The engineers of the Liverpool docks have used concrete in salt water for more than 40 years, and their experience has been that it will resist the action of salt water if properly made and deposited, *en masse, in situ.*

The Dock Department of the City of New York has also been using concrete in the sometimes salt, sometimes brackish, but always dirty, waters of the North and East Rivers for more than 40 years. An examination of the sea wall shows no deterioration, the concrete being as firm and solid as the day the blocks were set. This wall, however, was built of blocks manufactured on dry land, and not placed in the work until they had hardened sufficiently to be handled with safety. Each block weighs about 75 tons. As the concrete is protected, from a level 2 ft. below mean low water up, with a granite veneering, there is no way of determining how the material would have worn if it had been subjected to contact with the ice.

Pier No. 1 (New) North River, which was built about 40 years ago, rests on concrete piers and arches; only the outer faces are protected by granite. The arches under the dock, however, are badly pitted and scarred at about low water; this concrete was laid *en masse, in situ.*

In the early Seventies this Department had little success in depositing concrete under water directly in the work, and this method was soon abandoned; then the old scheme was taken up, of building the blocks on dry land and setting them in the work with a derrick.

A very interesting and instructive example of the failure of a concrete graving dock occurred in Aberdeen, Scotland. The dock was opened in 1885 and immediately showed signs of leaks, which gradually grew to be so bad that, in 1887, a thorough examination of the structure was made. Mr. Philip J. Messent, who was called in as consulting engineer, took samples from various parts of the dock, and in his report states that the deterioration existed chiefly in those sections where the proportions of sand to cement exceeded the ratio of two to one. In some places the mortar was a mixture of more than 1 part of cement to 3 parts of sand. Mr. Messent also found that

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the mortar plastered on the surface of the concrete, which was mixed in the proportions of 1 part of cement to 1½ parts of sand, had hardened thoroughly, and was in very good condition, as was also the concrete mixed 1 part cement, 2 parts sand, and 4 parts broken stone. The failure of this dock was due undoubtedly to the laxity of the inspectors on the work, who permitted the use of a mixture which, although composed of cement, sand, and broken stone, could not be called concrete. As the concrete was not impervious, the salt water penetrated it quite freely, and changed the composition of the cement, which resulted in failure. Samples were cut out of the concrete and sent to Professor Brazier, of Aberdeen University, to be analyzed. It was found that the deterioration was due to the chemical action of the salt water on the cement. Professor Brazier says:

"The analyses of the series of decomposed cement show a remarkable difference to the original cement, inasmuch as that in all these samples there is found a large quantity of magnesia, and a large proportion of the lime in the form of carbonate. I believe this alteration is brought about entirely by the action of sea-water upon the cement. There is no other source for either the magnesia or the carbonic acid."*

Every engineer should direct his energy toward obtaining an impervious concrete, especially in the case of graving docks; and, as it is possible to make concrete water-proof, and such concrete has proven to be the more lasting, the extra time devoted to obtaining this end is well spent.

The dry dock built by Luigi Luiggi, M. Am. Soc. C. E., for the Argentine Government, at the Naval Station at Bahia Blanca, was put in commission in 1902. This structure is perfectly tight, and has not shown any signs of deterioration or leakage.

The greatest care, however, was taken in making and placing the concrete in this work. Each delivery of cement was thoroughly tested before it was used. The proportions laid down by the specifications were strictly adhered to, and the joints between the various layers of concrete were carefully manipulated at the beginning of each day's work, so as to avoid all chances of leakage at these points.

In order to insure a good concrete, the proportions having been determined, the cement, sand, and broken stone must be carefully gauged and thoroughly mixed, so that each particle of stone will be surrounded by a coating of mortar. The mixing is best done by a machine, if large quantities of concrete are to be laid. There are many kinds of such machine on the market. All have their good points, and the engineer should decide for himself which mixer is best adapted to his particular work.

* The results of the investigations will be found in full in *Minutes of Proceedings*, Inst. C. E., Vol. CVII.

Mr. Davis.

The consistency of the concrete, again, is a matter of opinion and judgment, some engineers prefer it very liquid, others require it to be dense. In the latter case it is essential to tamp the concrete thoroughly after it has been laid in place, and the layers should not be more than 6 in. thick. In the former, the concrete will take care of itself, and the tamping is not so important. In either case, whether wet or dry concrete is used, great care must be taken to make a strong and tight joint between old and new work. To do this, the surface of the old concrete should be roughened by picking, and thoroughly cleaned and moistened; a layer of rich mortar, from 2 to 3 in. thick, should then be placed on the clean surface and the new concrete should be thoroughly tamped into the fresh mortar bed. In this manner, a good junction between the old and new work will be obtained, which will not cause trouble.

It is frequently necessary to lay concrete in cold weather. Although it is not advisable to do this, it may at times be necessary to push work to completion in spite of the low temperature. In order to prevent the concrete from freezing, some precaution must be taken, either by maintaining fires in stoves or by raising the temperature with steam coils, according to the location and magnitude of the work. On the other hand, it is just as important to protect the concrete from the direct rays of the sun, and prevent the water in the mixture from being evaporated during very hot weather, or the concrete will not set properly, will not gain its full strength, and the surface will be likely to crumble. Frequent watering of the surface of the concrete with a spraying nozzle, and covering the surface with moist sand, saw-dust, or canvas tarpaulins, will prevent the water from being evaporated, and the concrete will set properly.

Frozen concrete will in time thaw out and eventually set. The following may be of interest, as it shows what may happen to frozen concrete; although it is not intended to indicate by this experience that concrete should be allowed to freeze, even when being built in as large masses as the blocks mentioned. It was found necessary to move some concrete blocks, which were about 10 ft. 6 in. by 6 ft. 8 in. by 8 ft. and had been built in freezing weather. According to custom, the work of concreting was stopped as soon as the temperature in the moulds fell to 36° Fahr. The blocks had been completed and allowed to stand for 6 or 8 days, when it was decided to shift them to make room for more work. Chains were rove through the chain holes, and, as the attempt was made to lift the blocks, the chains cut through the concrete. It was found that small crystals of ice had formed in the concrete, and that the process of setting and hardening had stopped. The remaining blocks were left undisturbed until the following spring, when they were handled and set in the sea wall without further trouble. These blocks behaved well and showed

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no sign of having deteriorated, although built at the same time as the two which broke in handling during the cold spell. This experience does not indicate that concreting should be carried on during cold weather without some precaution being taken to prevent freezing, but that, if concrete, in large masses, once frozen, is left undisturbed, it eventually may thaw out and perhaps set properly. No samples were taken to test for hardness or porosity; in this case, weight was the main requirement.

It is impossible to expect good and permanent results if poor materials are used in concrete, even if laid with the greatest care, as was the case in the following illustration: A reservoir, part of a private water supply, was discovered to be leaking; in fact, it failed suddenly, and would not hold water. The conditions revealed by an examination made it all the more surprising that the structure was ever water-tight. It was built partly on rock and partly on a very hard clay. Its floor consisted of about 5 in. of concrete made with sand, gravel, and first-class American Portland cement. The sand and gravel contained a very large percentage of clay, loam, and foreign matter, nearly 50% of their ingredients being injurious to the concrete. The old floor was easily removed, being readily picked up; in fact, the clay under it was considerably harder than the concrete; its appearance at first glance was good, it was apparently smooth and hard; on a closer examination, however, a large number of soft spots were located, and the entire surface was found to be pitted.

The sand used was reported to have come from a sand bank which furnished the materials to be used in the repair work. This can readily be believed, as the sand and gravel, after being screened and washed, yielded about ¼ cu. yd. to the load of 1¼ cu. yd., and in one case 1 cu. yd. of sand shrank to about ⅛ cu. yd. after being screened and washed.

It is the use of such materials that condemns concrete, and probably is the cause of most of the failures which are reported from time to time. Absolutely clean materials should be insisted on, and none other permitted to enter into the mixture placed in the work; in no other way can satisfactory results be obtained.

If it is intended to use reinforced concrete, great care must be taken that the plans are properly carried out. It is essential to place the reinforcement just as the design shows it. Numerous failures can be attributed to placing the steel in the forms without any attempt to make the reinforcement conform to the drawings. To do this means care on the part of the inspector, and no concrete should be permitted to be placed in the moulds until the steel skeleton has been carefully examined, accepted as placed, and passed on by the inspector. This is essential, especially when one considers the carelessness of carpenters, who make no attempt to remove from the forms pieces of wood, no matter how large. These, being enclosed

in the concrete, naturally weaken the structure. It may or may not fall, but, usually, it will develop weak points. Mr. Davis.

In the concrete of a building which collapsed several years ago, were found, not only pieces of wood, but a large assortment of foreign matter, such as a pair of overalls which evidently had been placed in the form for safety, was forgotten, and eventually became part of the structure. It is such accidents, all due to carelessness, that have prevented a more rapid development of ferro-concrete construction in the United States.

Constant vigilance is necessary to insure good work and to obtain as nearly perfect results as possible. The entire work must be based on carefully drawn plans and specifications, and it is essential that they be followed faithfully.

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PAPERS AND DISCUSSIONS

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THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

Discussion.*

BY MESSRS. E. L. CORTHELL, W. H. WARREN, DAVID A. MOLITOR, SIR BRADFORD LESLIE, JOHN C. FERGUSSON, AND V. E. DE B. DE BROË.

E. L. CORTHELL, M. AM. SOC. C. E. (by letter).—There is no doubt that the spans of bridges are increasing in length and that there are many locations where the kinds of steel described and treated in this paper would be used to bridge over wide expanses of water not now possible, if they could be manufactured at a reasonable price. Mr. Corthell.

All engineers who are contemplating such structures will take great interest in the author's suggestions. It is to be hoped that the funds necessary for the suggested series of experiments may be found. The results would be of use, not only in the United States, but elsewhere, such as, for instance, in the proposed cantilever bridge over Sidney Harbor, Australia, the main span of which is to be 1 600 ft. long and 94 ft. wide between centers of posts, for four railway tracks, main roadway, motor roadway, and footway. Mr. Bradfield, the engineer, is soon to be in America, and this paper will interest him.

The extra price of 2 cents per lb. asked by manufacturers for the proposed high-alloy steel would restrict its use, but if it was called for by engineers, in sufficient quantities to warrant its general manufacture, no doubt the price would be cut in two, as the author indicates.

The author's remarks on page 680† about live loads are quite important. He states that:

"The live-load stresses on the main truss members will never be quite as great as they are computed, because, first, the trains on the two tracks never advance together so as to produce maximum web

* Continued from May, 1914, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., for March, 1914.

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stresses; second, such trains are not likely ever to cover entirely the bridge or any individual part of it, except, perhaps, the central span; and, third, it is improbable that any load of cars, unless they be ore or coal cars, will ever be uniformly full and loaded to the assumed limit."

All of which is true and might well be taken into account in the actual design of long-span railroad bridges.

It is customary at present to allow for a train load of 5 000 lb. per lin. ft. (from 3 000 to 3 500 lb. per lin. ft., 20 years ago). Not many cars weigh (loaded) 5 000 lb. per lin. ft., and these are not often loaded to their full capacity. A car having a capacity of 100 000 lb. will weigh about 50 000 lb., and is 40 ft. long. This brings the load to 3 750 lb. per lin. ft., which is not much more than that allowed 20 years ago. Are designing engineers expecting still larger cars than at present? It would seem almost impossible to increase the loads per linear foot to more than those which now obtain.

Certainly safety should be the desideratum, but if the dead load of a bridge can be decreased materially by using judgment in the application of live loads in designing, a great deal of money would be saved, so much, in fact, that cases might arise where the difference in cost would make practicable a project which otherwise might be impracticable.

This, and substituting for carbon steel a steel of a "high, cheap alloy, suitable in every particular for bridges", which, the author says, "is now before metallurgists and the builders of large metallic structures," may reduce weights and consequent costs to such an extent as to be of immense advantage in the projects of railway bridges of long spans.

The author is right in his statement that "the onus is on the Engineering Profession to see that the necessary experiments are arranged for." It is to be hoped that the paper will receive the discussion that its importance and character deserve.

Mr. Warren.

W. H. WARREN, M. AM. SOC. C. E. (by letter).—This paper is of unusual interest to the writer, as it is proposed to build a cantilever bridge over Sydney Harbor of 1 600 ft. span. In giving evidence before the Public Works Committee of New South Wales, in May, 1912, he strongly advocated the use of nickel steel in the cantilevers and suspended girders, with carbon steel in the floor-beams and stringers, and also in the systems of bracing. He also directed attention to Mr. Waddell's paper* on "Nickel Steel for Bridges," as supporting his own argument that considerable economy would result in the use of nickel steel on account of its higher elastic limit, and the fact that it can be worked under all the necessary manipulations in the shop in

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

a satisfactory manner, and also that its other physical properties are at least as good as carbon structural steel. Mr. Warren.

If an alloy steel could be produced having an elastic limit of 70 000 lb. per sq. in., with its other necessary qualities suitable for bridge work, at a moderate cost, a great advance would at once become possible in the art of the bridge builder, so far as the design of long spans is concerned. The paper shows that such a material may be reasonably anticipated in the future, provided sufficient inducements are given to the metallurgist and the steel manufacturer, and the greater economy that would result in the use of steel having a still higher elastic limit is clearly indicated in the diagrams which have been worked out in such a painstaking and elaborate manner. The paper is most opportune, and it suggests and indicates the probable method of procedure in a line of research which, it is hoped, the metallurgist will seriously consider. The writer is of the opinion that members of the Engineering Profession all over the world owe a debt to Mr. Waddell for the notable papers which he has contributed on the design of bridges.

DAVID A. MOLITOR, M. AM. SOC. C. E. (by letter).—This most interesting paper represents many years of labor and experience, and, as a contribution to engineering knowledge, should be accepted with due appreciation and regard. Mr. Molitor.

The author's conclusions relative to the advantages to be derived from the use of "high-alloy" steels in bridges, both in point of economy and greater span lengths attainable, are true beyond question, and, as pointed out in the paper, the problem of finding a suitable cheap alloy steel is now before the metallurgists and the builders of large metallic bridges.

It might be said that this problem has always been before the manufacturers, and possibly also the metallurgists, at least since the construction of the Eads Bridge at St. Louis; but, what has been accomplished, and who is most interested in progress along these lines?

Aside from a few innovations, chiefly with the aid of alloys such as nickel, aluminum, chromium and manganese, to raise the elastic limit and ultimate strength, and titanium as a purifying agent, little has been accomplished to improve the quality of steel for bridge and structural uses since the introduction of the open-hearth process. However, the two great defects in all manufactured steel are segregation and piping, for which no remedy is known to mill operators except the costly and wasteful method of discarding from 15 to 40% of the ingot, and, further, scrapping all material showing defects in the finished product. If these defects were readily discernible, this method might insure good results, but at high cost.

Both segregation and piping are local defects; the former is always hidden in the finished material, and the latter is usually invisible except

Mr. Molitor.

when it appears as a noticeable surface defect. Hence, it follows that no specifications can be written so as to insure a high-grade homogeneous material, in spite of the most painstaking mill inspection.

Engineers prescribe both the chemical purity and physical requirements for bridge and structural steel, and require that a test shall be made from a specimen taken from each ingot, and on the results of these tests the metal is either accepted or rejected, reserving the right to reject such of the accepted material as may develop defects in course of manufacture. The presumption is that all material not rejected in the mill or at the shops is uniformly good and up to the specifications.

However, nothing could be more erroneous, and, even if every single piece of rolled material were tested in like manner, this could not be accepted as absolute proof of quality. For there might be a hidden flaw or pipe, or spot of segregated metalloids, anywhere in the finished piece, which would never be revealed by any test, but might ultimately come to light through failure.

A careful perusal of the Report of the Committee on Rails, of the American Railway Engineering and Maintenance of Way Association,* will convince the most skeptical of the truth of what has just been stated.

Although segregation and piping defects are likely to occur in any or all material, they are less likely to produce disastrous results in bridges than in rails, owing to the more strenuous service of the latter. It is for this reason that ingots for rail steel are cropped about 30%, on the average, and bridge steel is cropped about 20%, on the average, with a minimum of about 12 per cent.

It should be noted that Bessemer steel is less subject to piping than is open-hearth steel, and that the latter suffers less segregation than the former, owing to its lesser sulphur and phosphorus contents and lesser quantity of metalloids.

Although these conditions exist in bridge steel of the present day, their deleterious effects are largely overcome by using moderately low working stresses. It is well, however, to reflect on the conditions which would inevitably follow when the same basic material is raised to higher strength by using an alloy or higher carbon contents. Unless segregation and piping can be effectually eradicated, it would seem quite out of the question to consider high-alloy steels safe for large bridges, as defects of this class would become far more serious in slender sections with high unit stresses. In other words, a condition which might be tolerated in medium steel, might become prohibitive in the high-alloy steel.

The crucial point of this discussion rests on the willingness and on the ability of the steel manufacturer, first, to improve the quality of the present-day product by the prevention of segregation and internal

* Vol. XII, Part 2, 1911.

pipes, and second, to raise the elastic limit and ultimate strength of this homogeneous steel by the use of alloys, taking care to secure proper ductility in the high steel. Carbon might be used to a certain extent, but, in all probability, at a considerable sacrifice of ductility. Mr. Molitor.

From the manufacturer's viewpoint, however, innovations of this nature do not interest him so long as his experience is profitable and he can dispose of his entire output without difficulty. The operating department of a steel plant aims merely to produce tonnage, and the commercial department seeks to dispose of this tonnage to the best advantage.

Albert Lucius, M. Am. Soc. C. E., at the close of his discussion* says:

"The engineer must get next to the metallurgist. It is highly probable that any metallurgist who knows how to produce a better steel alloy at commercial value will succeed very promptly in getting it on the market, * * *."

This is quite an erroneous view, for it rests entirely with the operating manager of a steel plant whether or not he will interest himself in what the metallurgist has to offer, and the metallurgist, in turn, is not getting things on the market. In other words, the metallurgist, who is appealed to as the savior in this matter, is quite helpless to do anything further than possibly say he knows how the task might be accomplished.

In all the learned discussions precipitated by investigations of rail failures, the conclusions reached, by metallurgists and others possessing a wide experience in testing and inspecting steel, are that practically all the failures are due to piping and segregation. However, the only remedy proposed by our learned friends, to overcome these serious defects, is to continue cropping off the ingots until the remaining portions appear to be uniformly good and sound. These same experts when advised that the difficulty could be overcome successfully by a very simple and cheap process which would be communicated to them if they were interested, failed to go further than to acknowledge the receipt of the writer's letter of more than a year ago.

This is a fair sample of the interest manifested by those who are most in the public eye.

It may add to this discussion to enter somewhat into detail as to the phenomena of piping and segregation, as comparatively little is known with certainty, and, hitherto, all attempts to overcome these defects in steel have proved only partly successful.

Although chemical reactions of a most complex nature are known to take place between the various impurities, sulphur, phosphorus, silicon, and oxygen, with iron, manganese, and carbon, forming compounds which are known as metalloids, it is practically impossible to show the exact nature and composition of these metalloids by chemical analyses. It is also quite probable that the metalloids in the molten state are very different from those in the frozen condition. There is also a consid-

* *Proceedings*, Am. Soc. C. E., May, 1914, p. 1615.

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erable gas formation in the interior of the molten metal, chiefly caused by the oxidation of the carbon and some of the impurities.

It is also known that the metalloids have a lower specific gravity than iron, and that their melting points are lower than that of iron, which being the case, the metalloids will cool more slowly than iron, and will naturally free themselves, to a certain extent, and on account of the difference in specific gravity will be forced laterally and upward into the lake of molten metal and continue in this action, so that the last quantity of metal that is cooled will contain a concentration of these segregated elements. The gases are similarly displaced, and, being extremely light, will hasten the collection of the metalloids into nodules which will travel upward along lines of least resistance as the metal cools in the mould. It is this effect, due to the difference in melting points, and the gravity displacement of the metalloids and gases, which gives rise to segregation and may, therefore, be classed as a physical phenomenon.

Piping is also a physical phenomenon, being caused by the shrinkage of the metal in the mould as it cools, and is the result of the following conditions: The metal is poured into the cold mould, and, as it strikes it, it chills or freezes to it at the bottom and sides, the chill being continued to the top level of the poured metal. The cooling or freezing takes place successively toward the top and center of the mould, and as the metal cools it naturally contracts and draws its supply for contraction from the molten lake in the center which sinks as the demand is made on it by the contraction of the surrounding metal, the result being the formation of the funnel-shaped cavity which frequently reaches below the center of the mould. This cavity may contain, in some instances, hanging walls or bridges, which may seal across the lower part of the pipe, resulting in a hidden or concealed pipe. This may not be detected, either in the bloom or in the finished product.

Segregation and piping are thus seen to be physical manifestations which cannot be prevented by any chemical process or treatment.

Although a chemically pure steel would necessarily be free from segregation, yet no steel, or, in fact, no metal, can be poured into an ingot mould without showing a pipe. The great necessity of freeing steel from internal pipes is thus made apparent.

The use of titanium unquestionably improves steel, and especially when added to Bessemer steel, which is high in sulphur and phosphorus, and also in gases. For this reason, Bessemer steel boils more in the mould. The addition of titanium is especially useful to quiet the metal by preventing the gas production through combinations with nitrogen and oxygen, forming light substances which rapidly rise to the surface of the ladle, purifying the steel at the expense of the titanium, very little of which remains in the steel. Hence, titanium somewhat reduces segregation, but does not affect piping.

Mr. Molitor.

More than a year ago the writer's attention was drawn to a process of casting ingots, by which the gases are all expelled, the metalloids are prevented from rising, and the steel cools gradually from the bottom upward instead of inward from the sides of the mould, thus preventing unequal contraction along different elements vertically in the mould. This is all accomplished by one operation, without loss of time in casting ingots, and at little or no expense outside of the apparatus on which the mould is placed while making the ingot casting. The process is one of forging the liquid steel by imparting vertical vibration to the mould and its contents, and is called by the inventors, Messrs. Maxwell and Lash, "The Liquid Forged Steel Process."

From the foregoing description, and the phenomena attending ingot casting, it is quite clear that segregation and piping can be completely obviated, thus making it possible to produce a perfectly uniform and homogeneous steel in which the ultimate strength and elastic limit are raised about 20% by the de-gasifying effect of the process.

The loss due to necessary discard of the ingots for bridge steel is reduced from about 20% to 3 or 4%, and, for rail steel, the reduction is from an average of 30% also to 3 or 4 per cent. This would result in an actual saving of from $1.50 to $2.00 per ton for steel production; the cost is merely an interest charge on the value of the machinery used and a small royalty on the patents, to say nothing of the far greater value of the finished product and the savings brought about by lesser rejections of the rolled steel.

That these statements are correct cannot be disputed when the tests which were made last year by the patentees are examined. Hence, the introduction of this process into the large steel mills would be somewhat of a revolution in steel manufacture as well as in the vastly improved quality of the finished product.

In the present grades of steel, defects due to segregation and internal or concealed pipes represent losses to the customer, and the visible pipe is a direct loss to the mill through discard. Hence, it is, to both customer and mill, that the introduction of this new process commends itself, and both parties should be equally interested in seeing it introduced.

However, it is more than likely that the demand for the better steel must be first created before any mill will undertake the production of such an article.

To the writer's mind, there is no question regarding the possibility of producing high-alloy steels, of the kind sought by modern bridge builders, whenever it suits the convenience of the mill manager to disabuse his mind of several preconceived views and to express his willingness "to be shown". The writer also ventures to add that aluminum, possibly with nickel, will offer the most desirable combination with steel, owing to the great ductility it imparts.

Returning to questions of design, it should be observed that our

Mr. Molitor.

present methods would suffer numerous modifications when dealing with high-alloy steels, even in small bridges, as our knowledge of secondary and impact stresses would require. Deflections due to moving loads would be increased owing to the higher unit stresses used and the practically constant extensibility of all grades of steel. Secondary stresses might not suffer much increase on this account, because the connections between slender high-steel members would be less rigid, though impact stresses might be increased because the ratio of live load to dead load would be increased along with the greater flexibility of the structure.

As discussions of this kind are sometimes fruitful in bringing about advancements in engineering science, the writer expresses his high appreciation of the author's efforts in the direction of progress, and closes with the optimistic feeling that some substantial gain in the art and science of bridge building may soon come as a reward, provided the steel makers will do their part.

Sir B. Leslie.

SIR BRADFORD LESLIE (by letter).—In India the river beds are generally alluvial for an unknown depth, and piers of bridges having to be sunk to a great depth are very costly, consequently, it is economical to use spans of considerable length, but the writer is not aware of any cases in which exceedingly long spans are likely to be required. Nevertheless, the paper is of the greatest interest to bridge engineers.

The author has fully established the necessity for experimental investigation of the problem of discovering a high-alloy steel which can be subjected to heating and working at temperatures attainable in the ordinary operations of bridge construction, without impairing its strength, and which can be placed on the market at a price moderately exceeding that of the ordinary carbon steel of commerce. Such a metal would greatly reduce the weight and cost of girder bridges of moderate spans. The percentage of nickel required is so small that it seems probable the ores will be discovered in sufficient abundance, and, if so, the demand for the material must soon result in the practical realization of the author's far-sighted ideas on bridge construction.

The writer belongs to the iron age, which is ancient history, but can well remember the relief afforded to bridge designers by the increase of 50% in the elastic limit when carbon steel superseded iron as the material for bridges, and he hopes he may live to see the further improvement that will result from the general use of high-alloy steel for that purpose.

Although the writer feels that he is not qualified to offer useful criticism of the investigations described in the paper, he desires to state that his experience ranks him among the unbelievers in the dangers of so-called "impact". If that bugbear can be dismissed, it will effect great economy in girder construction. He is equally skeptical as to the supposed injurious effects of alternating stresses which do not exceed the elastic limit.

Sir B. Leslie.

A train does not drop on to a bridge, it rolls on and off; at whatever speed it may be traveling, the pressure is applied and relieved gradually—a mass falling by gravity moves but slowly when it begins to fall—any blow there may be is due to the action of the springs. Velocity in its application does not affect the stresses distributed over a given area of material by any load. At 60 miles an hour the axle of a vehicle may be subjected to alternating stresses at the rate of 10 per sec. but not to "impact"; properly lubricated, the axle keeps cool. The worst "impact" that axles get is at worn rail joints which strike a blow felt through springs and cushions, and is the principal cause of axle failure. "Impact" is locally and not otherwise destructive. If a bar of iron, such as a 6-ft. length of rail, be suspended by one end and constantly struck four or five blows per sec. by a hammer, heat will be evolved by the superficial destruction of the metal; such destruction will be local and cumulative; the entire bar will vibrate; but, however long continued, the vibration will not affect the properties of the metal. In a similar manner, the top of a pile is punished by hard driving and the remainder of the pile suffers no injury, though it is true, the pile receives but a few hundred blows.

If bridges suffer from "impact", it must be transmitted through the rails. Rails are worn out by attrition; excepting at worn joints, there is no "impact"; millions of loaded axles rolling over rails at high velocity do not impair the strength of the metal.

Many years ago the writer made careful experiments on scrap rails before using them for structural purposes*. The rails were of iron, of the old double-headed section formerly in use in England. Pieces were cut from both heads and tested under tensile and compressive strain, also for ultimate extension and contraction of area. In no instance was it found that the scrap metal had lost any of its original properties, under millions of tons of traffic resulting, not in "impact", but in constantly alternating stresses. "Impact", indeed, there is, at the joints, causing failure of the ends of the rails.

Prior to the last 25 years, no attention had been given to "impact", and no account had been taken of it in designing girders. The writer has never met with any authentic instance of failure on that account. On the contrary, many large bridges, both of iron and steel, are now carrying axle loads in excess of what they were originally intended to carry, without any indication of failure, except in the platform system, where increased panel loading has, in some cases, involved strengthening.

The only precaution advocated by the writer to mitigate "impact" (the effects of which in short spans are injurious to the masonry abut-

* Owing to heavy rates of freight from India to Europe, scrap material which could not be rerolled or worked up in India, in about 1878, was practically unsaleable, consequently its use for structural purposes, instead of importing new material, was very economical.

Sir B. Leslie.

ments, rather than to the girders themselves), is to build bridges up to, say, 100 ft. span to carry the ordinary track laid in good ballast. This facilitates maintenance, prevents the passage of the bridge being felt on board the train, and the shaking of the masonry abutments, and saves the necessity for allowance for supposed "impact". Such saving is discounted by the girders having to be provided with flooring, and built strong enough to carry ballast. However, there can be no comparison in the comfort of traveling over a ballasted bridge carrying the ordinary track, and running over a rattling, noisy structure in which the track is fixed directly to the girders.

The writer infers, from the paper, that it is customary to build long-span bridges for double track, presumably because the increased width between the girders relieves the bending moments due to wind pressure, and improves the lateral strength of the structure generally.

If worked on a proper system, a single-line bridge will accommodate a very heavy traffic. Two trains following each other every 10 min. gives twelve trains in each direction per hour. Where intensity of traffic absolutely demands it, the expense of a double-line bridge must be met.

Instances have occurred on the 5 ft. 6-in. gauge railways in Bengal of trains being brought to a standstill and overturned by wind pressure. This indicates a side pressure approaching 40 lb. per sq. ft. over considerable areas. No such accident has hitherto occurred on a bridge in India, though it is probably what took place at the collapse of the Tay Bridge in 1869, and it is necessary, therefore, to take account of the possibility.

It is suggested that all important bridges should be equipped with a reliable anemometer interlocked with the signals, so that whenever the wind attains a velocity exceeding, say, 80 miles per hour, or 17 lb. per sq. ft., the traffic over the bridge would be stopped; considering the risk, this is only a reasonable precaution. Thus, in the case of wind pressures exceeding 17 lb. per sq. ft., the effect on the bridge would be limited to that on the structure only, below that it would be necessary to take account of the wind pressure on the train as well as on the bridge itself. Such an arrangement would enable the economy of single-line bridges to be realized wherever practicable and desirable.

Doubtless, however, all these points have received due consideration in the preparation of the paper. They are all the observations that the study of this valuable paper, considered in the light of 60 years' experience in bridge building, enables the writer to offer.

Mr. Fergusson.

JOHN C. FERGUSSON, ESQ. (by letter).—The amount of time and energy the author has devoted to his subject, and to the preparation of his valuable diagrams, showing, by comparison, the advantages of using steel of higher tensile strength in long-span bridges, shuts out all but the most friendly criticism. Yet facts are stubborn things;

we do not look at them from the same plane, so we cannot all see alike. It is this very difference of view which, in the long run, gives the real value to the exchange of ideas among civil engineers, by bringing things into true focus. Mr. Fergusson.

The use of nickel steel cannot be adopted, generally, as the best material for all classes of bridgework; its particular advantages adapt it more for use in bridges of very long span than in short ones. Its special advantages—elastic limit and tensile strength, hence lightness—over the best classes of open-hearth carbon steels, are only obtained in nickel steels of the very highest grades; and their cost would be prohibitive in ordinary structures. Carbon steels, for a tensile strength of 40 tons per sq. in., have advantages over nickel steels carrying low percentages of nickel. The open-hearth carbon steel is a more reliable product, is stronger in compression, has greater resistance to impact, its manufacture and testing are extremely simple, and the main results are constant; whereas, the testing of nickel steel during the process of manufacture is very complicated, the material is subject to great variations; and, further, the combinations of carbon, manganese, titanium, and tungsten, even in the smallest fractional percentages, produce most unlike results.

It has been found that the addition of nickel to iron causes the nickel to act as a flux on the carbon, manganese, and other minerals in combination with the iron; and the anomalous results produced vary very much more owing to slight changes in the proportion of carbon, manganese, or other metals in the chemical composition of the ingot, than in the mere change of proportion in the parts of the nickel to iron.

The idea of adopting a standard specification for nickel steel, to be used in bridgework, seems to be wrong, because the conditions required for various spans and loads are so variable. Would it not be wiser for a civil engineer to state his requirements and see that they are carried out, leaving the actual manufacture of the metal to the metallurgists and ironmasters?

To use a nickel steel of very high tensile strength, chiefly to save weight in the bridge material—unless where the conditions are imperative—would seem to be extravagant, and even unwise. There must be risk in trusting to the highest limit of tensile strength of a manufactured steel which is known to be very variable. The mere saving of weight in bridges of moderate span would hardly warrant the risk, coupled with the extra cost of material of higher grade. Standard specifications for nickel steel would be worthless unless they met the highest conditions and requirements for the longest bridges mentioned by the author. It is for bridges of this class, of exceptional length, that he hopes to find a sufficiently strong and suitable material in nickel steel.

Mr. Fergusson.

The author, in his previous paper,* states that the best percentage of nickel was found to be 3.5. The allowance of impurities in the nickel steel was put as phosphorus, 0.03%; sulphur, 0.04%; silicon, 0.04%, and manganese, 0.75 to 0.85%, which produced an elastic limit of 61 300 lb. and an ultimate strength of 99 300 lb. per sq. in. The impact was less than that of carbon steel, the relative values being 87 and 73 to 100, for low and high nickel steel.

The author also states that the nickel present in larger quantities makes the material unworkable in the shops.

If the author is still of these opinions, and if we may regard his present able paper as supplementary to his former one, the foregoing statements deserve consideration, because they do not agree with the general idea as to the chemical composition of a very high-class nickel steel that would best fill the requirements of the extraordinarily long bridges he has mentioned. The proportion of 3.5% of nickel, which he gives, appears to be too low. As he does not mention carbon in his analysis, it may be presumed that he has eliminated it; surely this is a difficult operation in commercial manufacture? This is a most important point, for, with a low percentage of nickel, the reduction of carbon below 0.20 becomes essential.

Provided the carbon is kept low, there is a constant rise in the tensile strength of nickel steel, in proportion to the increase of the nickel present, up to 12 or 15 per cent. The increase of nickel requires a decrease of carbon, together with an increase of manganese, otherwise, although the breaking load may be the same, the value of the elastic limit will fall rapidly.

The writer has pleasure in acknowledging that some of the earliest and most successful workers of nickel steel are Americans. The Bethlehem Iron and Steel Company, the Homestead Works, of Pittsburgh, and others, are well known.

There are a few points about which there appears to be considerable difficulty to agree. The author states that steel containing more than 3.5% of nickel was unworkable, yet in Sheffield they found no difficulty in working material containing nickel, 7.65%; manganese, 0.68%; and carbon, 0.17%, although it was hard.

In America there is a preference for adding nickel in iron ores, whereas in England it is preferred to add the nickel otherwise. In Sheffield it is found difficult to produce forgeable bars in the absence of manganese, and it appears to be impossible to get satisfactory results.

The writer ventures to point out two disadvantages in nickel steel for bridgework:

(*a*) The difficulty of welding, and working;

* "Nickel Steel for Bridges", *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101. Abstract in *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVI, p. 345.

(*b*) The necessity of keeping the carbon extremely low—under 0.02%—whereas the fatigue resistance of steel is increased enormously by raising the percentage of carbon. Mr. Fergusson.

V. E. DE B. DE BROË, ESQ. (by letter).—The writer has read this paper with interest, and begs to offer the following remarks. Mr. de Broë.

Spans.—As compared with Indian practice, the length of the spans required in America is remarkable. The Government of India railway bridge regulation tables are worked out for spans of from 5 to 600 ft., and usually, the engineers can negotiate big sandy rivers on deep wells with girders of from 150 to 250 ft. span. The writer believes that the only railway bridge in India of greater span than 600 ft. is that over the Indus at Sukkur, which is between 700 and 800 ft. Of small spans, ranging from 10 to 100 ft., Indian railways have a great number, and additional ones are being erected daily. It is readily understood, therefore, that, with engineers in India, the use of a metal of excessively high tensile strength is not of so much consequence as it is with American engineers, faced as they are with the problem of bridging vast clefts and chasms with single openings.

Strength of Metal.—In preparing specifications, the writer works to the "British Standard Specification for Structural Steel" (revised in August, 1912), which enjoins an ultimate tensile breaking strength for ordinary girder sections of "between the limits of 28 and 33 tons per square inch." As the elastic limit is generally not less than 50% of the ultimate strength, and good firms usually work to this without extra charge, it follows that:

$$\frac{\overset{\text{Tons}}{33} \times \overset{\text{Pounds}}{2\,240}}{2} = 36\,960 \text{ lb.}$$

is the extreme elastic limit per square inch reached in practice. The author states that ordinary carbon steel is usually furnished by American manufacturers with an elastic limit of 35 000 lb., or an ultimate tensile strength of approximately 32 tons per sq. in., so that in this respect, Indian engineers are on an equal footing, and it is not until American engineers begin to mix an alloy that they get a higher elastic limit.

It is noted that the author was successful, with French manufacturers, in realizing the high standard of 45 kg. per sq. mm., equivalent to 28½ tons per sq. in., or an ultimate breaking strength of 57 tons per sq. in. Nothing of this sort has ever been heard of in Indian practice. The author also expresses himself as sanguine of reaching in America 60 000 lb. = 27 tons elastic limit, or 54 tons ultimate strength; and his researches have led him to believe that even the colossal figure of 100 000 lb. = 44 tons elastic limit, and 88 tons ultimate tensile strength, is realizable by persistent and enduring experiment until the right mixture has been obtained.

Mr. de Broë.

The writer has not had the pleasure of reading the author's former paper, "Nickel Steel for Bridges," and is not chemist or metallurgist enough to form an opinion as to what can be done in the laboratory or in small melts in the production of an alloy greatly superior in strength to carbon steel; it is obvious, however, that should an alloy ever be produced approximating in strength three times that of steel (at a commercial market value), it is not only America that will benefit, but every railway in the world will be in a position to revolutionize its bridge stress and strain calculations with a view to economizing in weight of metal.

The estimated cost of the author's proposed experiments, \$100 000 = £20 000, or even twice this sum, as suggested, is a very large one for private enterprise, but not for a Government or a group of Governments to contribute, and it is further to be noted that, should the experiments result in the success which the author anticipates, the use of the new and stronger metal could and would be extended to other forms of construction besides bridge girders. There are no millionaire philanthropists in India, and so far as that country is concerned, anything of the sort suggested would have to be done by the State.

Cost of Steel.—On page 683* it is noted that the author is handicapped with the high cost of 4 cents per lb. for steel for plate-girder spans, or £18 sterling per ton. The highest price paid in England (for export to India) during the last few years, was £13/10/0 per ton, and, even adding sea freight, it would seem that, for ironwork required reasonably near the seaboard (and not involving long railway carriage to the site of the works), London would afford a cheaper market than America.

Impact.—Impact is a subject to which the writer has devoted a great deal of study of late years, and he has come to the conclusion that it may be totally ignored, provided a working stress not exceeding 8 tons per sq. in. for steel is allowed, and a factor of safety of 4 is used.

Of course, there is much diversity of opinion on this subject in the Profession. The Government of India enjoins the use of the Pencoyd formula:

$$I = \frac{300}{L + 300} = \text{a coefficient by which the live load is to be multiplied;}$$

where L = length of loaded distance, in feet, producing maximum strain.

This gives:

In 5-ft. spans 98% of moving load,
In 600-ft. " 33% "
As against 78% } as calculated by the author's formula,
and 36% }

$$I = \frac{40\,000}{L + 500}.$$

* *Proceedings*, Am. Soc. C. E., for March, 1914.

Mr. de Broë.

The writer has recently come into possession of the Swiss Government revised regulations (issued in 1913), in which the formula for impact is:

$$2(15 - l) \text{ per cent. of the moving load,}$$

l being the span, in meters.

It is obvious from this that no impact allowance is enforced for spans of more than 50 ft., and the percentage works out: For 5-ft. spans 27% only, as against that of the Government of India, 98%, and the author's 78 per cent.

The Swiss working stress is 6.98 tons per sq. in. for steel.

The Board of Trade (England) fixes a working stress of 6½ tons for steel, which is intended to cover impact allowance. Mr. A. H. Shield, writes as follows:*

"The limiting stress of 5 tons per sq. in. for which the early bridges were designed did, in fact, provide for impact, and was fixed by the Board of Trade in 1859, some years after experiments had been made by a Royal Commission, appointed to consider the application of iron to railway structures.

"In the course of these experiments the additional strain upon bridges due to a rapidly moving load was measured by the increased deflection. A fixed permissible stress obviously failed to graduate the impact according to the span, and the allowance made in fixing 5 tons was probably insufficient for spans under 20 ft., for which at that time the use of wrought-iron was, however, uncommon. The point which it is important to bear in mind, is that the old standard admissible stresses, 5 tons for wrought-iron and 6½ tons for steel, did include an average allowance for impact, and that this should be taken into account in making a more accurate provision according to the span, or more properly, according to the shortness of the period during which the load comes upon the bridge or part of the bridge under consideration. It is only if this is neglected that the application of a graduated allowance for impact leads to a general increase in the weight of bridges.

"The sufficiency under average conditions of a factor of safety of four without further allowance for impact, is substantiated by experience in this country for 55 years; and any rules which require a greater allowance under similar conditions are indicative either of a desire to provide indirectly for the growth of the load to which structures may be subjected, or to provide a wider margin against errors of design and workmanship than has proved to be necessary under the conditions of responsibility and supervision that have obtained in the past.

"Whichever may be the motive of the authority responsible for the increased burden it is equally deserving of the attention of those interested financially in the promotion and construction of railways in our colonies and dependencies."

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* *Railway Gazette*, June 12th, 1914.

Mr. de Broë.

The writer has not space or time to state exhaustively his reasons for recommending the total exclusion of the impact factor in bridge calculations, and perhaps the author would not thank him were he to enlarge on the subject, for a full exposition of his views would cover many sheets of paper. Briefly, however, he may say, first, that he does not think deflection tests (on which many of these formulas are based) are reliable. The following is from the Government Inspector's report on two spans of 40-ft. girders which the writer's company recently erected in Southern India:

"Two spans were tested with a B-class engine, one a dead load test and the other at 45 miles an hour. The deflection in both cases was 0.15 inch, which is very satisfactory."

This is one of the many instances.

In the second place, if a diagram is plotted, in the shape of a column (Fig. 27), and divided into three lengths represented by:

(1) A line across at, say, 8 tons (below which the girder may be loaded);

(2) A line across at 20 tons (below which is obtained an area of safety, which may be encroached on occasionally); and

(3) A line across at 32 tons (below which is represented the danger zone, never to be encroached on);

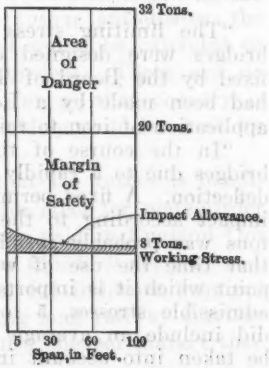

FIG. 27.

we may see how very small is the margin of risk if, above the 8-ton line, we hatch in the professional allowance for impact. We may also see how large is the margin of safety and how remote the area of danger.

In the third place, the various working stresses allowed by different authorities, and the factor of safety of four, are in themselves wholly empirical, and the writer sees no logical sequence in building, by the application of quasi-scientific formulas, on an empirical foundation. Nevertheless, he would be very interested to see the "more accurate formula" which the author has evolved from deflection experiments, mentioned on page 669,* and the data on which it is based.

* *Proceedings*, Am. Soc. C. E., for March, 1914.

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PAPERS AND DISCUSSIONS

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DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

BY MESSRS. ALEXANDER C. HUMPHREYS, J. J. H. MUIRHEAD, CHARLES KIRBY FOX, T. KENNARD THOMSON, W. W. K. SPARROW, J. P. SNOW, E. W. JAMES, GEORGE L. DILLMAN, AND WALTER S. WHEELER.

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ALEXANDER C. HUMPHREYS, M. AM. SOC. C. E. (by letter).—This report is dated December 1st, 1913. The Committee reserved the right to make insertions, additions, etc. The report was presented at the Annual Meeting, January 21st, 1914, and was then referred back to the Committee for fuller consideration and to afford opportunities for further discussion.

On February 28th there was issued an "addition" to this report, written by Chairman Stearns of the Committee. In this addition the report itself is styled "The Progress Report, etc." The additional matter presented appears under the sub-heading: "Preliminary Discussion of Depreciation;" and notes attached to this statement show that it was issued in anticipation of the discussion to be had at the meeting to be held for that special purpose on March 11th, and that this addition to the report was for the purpose of "further discussing the matter [depreciation] in this report in a very elementary way." Another special meeting for discussion was held April 2d, and it is to be hoped that other meetings will be held before the report is finally presented for action.

In discussing this important report, which the writer proposes to do in some detail, he wishes at the outset to make two general statements of belief:

First, the valuation of a public utility, and particularly of a railroad, does not offer a complete solution of the rate-fixing problem.

Second, the "principles and methods for the valuation of railroad property" are the same as for the valuation of "other public utilities"

*Continued from May, 1914, *Proceedings*.

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only in a general way, and this report applies more particularly to public utilities other than railroads.

The writer has made an earnest and extended study of the report from the viewpoints of the engineer, the operator, the accountant, and the investigator. During all his forty odd years of experience in these four departments he has been making a particular study of "depreciation", so-called. Then, if he has failed at any point to get the meaning of the report, it has not been through lack of effort. This is in answer to the general suggestion made by Mr. Stearns at the meeting on March 11th.

The subject of the report is of vital interest to the people as a whole. If we keep in mind the fact that investors are not a class by themselves, but include presently or prospectively all those within our borders of frugal mind and intentions, we cannot escape the grave responsibility of participating in this discussion only with open minds.

Many of the reports and papers presented to professional societies fail wholly or in part to develop the truth because of lack of time and effort devoted to the discussions; let us see to it that this charge cannot rest against us in this case.

The writer finds much in the report with which he completely agrees, and much with which he can in part agree. He is sorry to say that he finds other parts with which he completely disagrees.

If the "principles and methods for valuation" as set forth in the report were adopted as a whole, the result would work grave injustice to the honest investor. The more the writer studies the report, the more he is impressed with the practical quality of certain of its parts and the excessively academic and non-practical quality of certain other of its parts. The treatment of "overhead charges" comes within the first classification, while the treatment of "depreciation" comes decidedly within the second classification.

The questions involved in the valuation of public utilities, to be solved fairly, must be solved practically. Unfortunately, these valuations appear to present many suggestions for solutions of the most academic character.

The writer will not hesitate to repeat himself; first, to meet the points as they are made in the report; and, second, to minimize the chance for misinterpretation—an ever present danger—by discussing from different viewpoints the questions as raised.

Page 6.—It is true that "during the year 1912 and since, several extended and valuable works upon this subject [valuation] have issued from the press." Mr. Robert H. Whitten's "book of nearly 800 pages" is particularly commended. The writer trusts this does not mean that all of Mr. Whitten's "clear and concise discussions" are recommended for acceptance. Some of the books referred to are more "extended"

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than "valuable," and particularly those written by men without actual experience.

The writer agrees that "valuation * * * must * * * be made in conformity with existing laws", but he does not agree that this necessarily means conformity with the decisions to date. In fact, this would be an impossible condition to apply, varying so radically, as do many of the important decisions. The Committee recognizes this in the statement: "the whole subject is in a developmental stage."

There is great difference of opinion as to what constitutes a "fair return". What would be a fair return in one case would not be a fair return in another. In the New York Consolidated Gas Case, the U. S. Supreme Court unanimously decided, in that particular case, because of the advantages enjoyed, or said by the Court to be enjoyed, that 6% was not confiscatory. This decision did not state that this was a fair return. This decision has been misquoted and misapplied frequently. We cannot depend safely on fair sounding words as to what constitutes a fair return.

Page 7.—Undoubtedly, the service rates should provide for an allowance for so-called "depreciation." The expenditures for renewal of plant is one of the losses of the business or costs of the service, and should be so treated. The writer does not agree, however, that in all respects "the principles are similar to those involved where one man loans money to another." The lender is not obliged to keep his money loaned to the borrower after the loan falls due. The public service corporation is obliged to keep its investment in the plant indefinitely.

The writer completely agrees with the statement that "The public may be as injuriously affected by too low as by too high rates;" but it will be difficult and perhaps impossible to so convince the public, the commissions, and the Courts, as long as engineers of standing are so wide apart as to the fundamental principles of "valuation for the purpose of rate-making." If regulation of public utilities results in the fixing of rates so low as no longer to attract new capital, the public must suffer. This is not a theory but a condition already in evidence.

Page 8.—The writer does not agree with the Committee that we should acquiesce in the action of the commissions and Courts when they announce decisions "without disclosing definitely the method by which the conclusion has been reached." Nor should we necessarily concur in the opinions of the commissions and Courts. They should be treated with due respect, but it should be borne in mind that "the whole subject is in a developmental stage," and hence the Courts yet have to be educated in the questions involved, and we cannot help in that process, as we should, unless the commissions and Courts disclose their mental processes.

Page 9.—The writer is unable to find that "deductions for depreciation" are necessarily involved in all the Court decisions. This refers

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to the "depreciation" which includes obsolescence, inadequacy, and physical decay, which is to be eliminated periodically by renewals of plant.

The writer agrees with the statement:

"The public should not be required to pay a return upon extravagant or injudicious expenditures; on the other hand, the rates should secure to the owner a fair return on his reasonable outlay."

However, we must not lose sight of the danger here disclosed. There will be opportunities for wide differences of opinion as to what is a "fair return" and what is a "reasonable outlay." The writer would not be surprised if there might be found, among the members of this Committee, a not inconsiderable difference of opinion on these two questions, in cases widely varying as to conditions. Consulting engineers, who, perhaps, have not had to face deficits as a consequence of their estimates, would be likely to have quite a different opinion as to "reasonable outlay" from the engineer-operator who has had to make good on his estimates.

Page 10.—Referring to the paragraph in the middle of this page, and to many other portions of the report, it is to be regretted that the Committee has not carefully and completely defined what is intended by the term "net depreciation." It will be clear what the writer means by depreciation before this discussion is completed. "Depreciation" is a term used loosely. The Committee here seems to differentiate between "depreciation" and obsolescence, though in other parts of the report it would appear that obsolescence is to be included in "depreciation" allowance. If we intend "depreciation" to cover deferred renewals, necessarily, obsolescence and inadequacy are included in addition to physical decay or exhaustion.

Page 11.—

"A valuation based upon book accounts can be kept up to date by adding to the original valuation the cost of additions and replacements made from time to time to the property, and by deducting the amounts which the corporation is permitted to earn annually to offset the net depreciation of the property."

As a mathematical proposition, this statement of the Committee would be correct provided the "net depreciation" were ascertainable by estimates made in advance; but this "depreciation" as estimated depends on many assumptions which must wait on the facts for confirmation or correction. At the best, therefore, it is only an approximation. Furthermore, the "depreciation" does not necessarily indicate any true depreciation as tested by the quantity and quality of service capable of being furnished by the plant thus depreciated. "The amounts which the corporation is permitted to earn annually to offset the net depreciation of the property" should be from the first appropriated to the renewal or replacement of plant. The corporation thus

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acknowledges its liability therefor, and there is no sound reason based on ethics why, pending these renewals which are necessarily of the future, the property should thus be lessened in book value in connection with rate-making. The question of rate to be charged should primarily be determined by the character of the service rendered. If the plan proposed should result in always showing a net deduction for "depreciation", as it would do in most cases, then the property might be subjected to confiscation, although in a varying amount; and if a deduction is made in cases in which the investment is thus subject to confiscation, then the amount of this deduction should be included in the original cost as an item of overhead charge. Here there would be a further difficulty, for the reason that the measure of this overhead charge, as affecting rate-making, would be a variable quantity. This variability of the amount of "net depreciation" of itself suggests the injustice of deducting therefor.

The original cost of the property should be considered in the first place as an asset. The cost of maintenance and renewal of property should be charged to a loss account. An asset account in the progress of time may be found to contain items of loss, but we need not start out to complicate unnecessarily the keeping of the accounts.

Apart from the question whether or not estimated "depreciation" should be deducted from plant valuation, the information which the Committee is looking for can be secured in better form, and in a form more available for analysis, by crediting the "depreciation" annual allowances to a renewal reserve account and later debiting to this account all the expenditures for the renewals which are contemplated in the scheme through which the "estimated depreciation" is computed. The credit balance of this account would then show the accrued "depreciation", and this would also show the avowed liability of the corporation on account of deferred renewals.

Page 12.—The writer fully agrees with the Committee that the cost-of-reproduction should not be based on a "hypothetical property of modern design, capable of producing the same results [or so claimed by the designer] as the plant to be valued."

Pages 13 and 14.—"Unused Property."—The writer agrees with the Committee that deductions should be made for surplus capacity "only in extreme cases, if at all."

As to property not actually in use at the time of valuation, it has not infrequently happened in the writer's experience that such property has been omitted improperly from the inventory; whereas, if the inventory had been made by an expert operator, it might have been evident that this portion of the plant would be required at times and perhaps for a majority of the months of the year.

Property should be abandoned in the interest of economy and good service, and every advance toward more economical and efficient man-

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agement should be encouraged. If the profits of a company are to be limited to a low return (perhaps arbitrarily declared by some to be a "fair return") on the property actually in service at any given time, the incentive to secure the best management is weakened, if not destroyed. Under this condition, and while facing a valuation for rate-making purposes, no company could afford to abandon property in the interest of economy and good service. Any scheme of valuation that does not include property which has been abandoned in the interest of economy or good service or both, and which has not been paid for out of earnings, is defective, unfair, and against the interest of the public utilities and the public.

Page 15.—"Present or Original Conditions."—In justice, a public service corporation should be entitled to a fair and adequate return on the legitimate investment in its property. If such investment can be determined, assuming that it has been properly and intelligently made, it should not be necessary to make a valuation.

In the majority of cases it is not possible to determine the investment, and so, at present, under Court rulings, a valuation appears to be necessary. This valuation is intended to serve as a basis of rates for the future, and therefore should be made as of the present, and not on the basis of conditions of the past. In the case of the prices of copper, iron, and other materials, which fluctuate through a wide range, it is fair to assume that the future will, in this respect, as to fluctuations, be a repetition of the past, and, therefore, average prices through a term of years may fairly be used.

It is also necessary to study the events of the past to see in what condition the real estate, right of way, and other elements of the property would be, if the utility did not exist. After determining as nearly as possible the conditions that would control if the utility were not in being, an estimate should be made of the cost of reproducing the property under such conditions.

It is no more justifiable to deprive the stockholder of the unearned increment of increase in the value of his property than it is to confiscate the increase of value of neighboring property. This applies equally to real estate and to the increase in value of underground structures due to subsequent paving. The question is not what it would cost to reproduce the property under conditions varying at different times, but what it would cost to reproduce it under such conditions as would exist at the time of valuing if the property did not exist.

It may here be noted that in the recent Kings County Lighting Case, the New York Public Service Commission was over-ruled by the Court, and the cost of subsequent paving was allowed. This decision, in turn, has been reversed—another instance of disagreement among the judges on questions of principles.

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An estimate of cost of reproduction as a whole should not also include the additional cost of piecemeal construction. On the other hand, such cases as the New York Central Terminal serve to show that frequently it is necessary to make full percentage allowance for omissions, contingencies, and other items not represented specifically on the itemized inventory. Those who have had any considerable experience with engineers' estimates should know that bankers and business men have had too much reason for regarding these estimates with suspicion; and hence the more cautious of these guardians of the purse have been found adding considerable percentages to provide against the engineer's failure to provide adequately for the omissions and contingencies always encountered in constructions of any magnitude. Here the Profession as a whole has suffered through the lack of experience, the carelessness, or the narrowness of view of some of its members.

Page 18.—"Application of Reproduction Method of Valuation."—Though the writer disagrees with the Committee as to deduction for "depreciation" due to age, he is fully in accord with regard to inclusion of development expenses and working capital. He is also glad to note that the Committee draws attention to the fact that these and other items of construction cost are sometimes erroneously classed as intangibles.

Page 19.—"Inventory."—The Committee very properly draws attention to the fact that an inventory of an existing property does not necessarily, or even probably, give a complete schedule of the items of cost which went into the construction.

Page 20.—"Unit Prices."—In establishing the reproduction cost of a property, on first thought it would appear that, to be consistent, the unit prices prevailing at the time of valuation must invariably be used; but further thought argues for an acceptance of the view of the Committee:

"That the prices should not be fixed with reference to occasional fluctuations in the prices of labor and materials but rather at the general or prevailing prices for labor and materials, which, as a rule, can be deduced by taking the average or prevailing price for a term of years preceding the date of the valuation."

The writer agrees also that "an evident trend toward higher or lower prices" should be taken into account. It should not be forgotten that the valuation is being made with regard to the rates to be charged immediately in the future.

Page 20.—"Overhead Charges."—The writer wishes to endorse with all possible emphasis the opening statement of this section:

"When the items contained in the inventory are multiplied by the corresponding unit prices and the products are added, the result is only a part of the cost of creating the physical property, for the reason

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that the unit prices used are generally based upon the direct cost of similar works, and do not include the indirect or incidental expenses."

Again:

"In order to obtain the full cost of a building, it would be necessary to include the fees of architects, the amount paid inspectors, the cost of grading the grounds, the amounts expended for interest, taxes, insurance, and legal expenses, and a fair sum for the value of the time devoted by the owner to the planning and construction of the building."

Again:

"The allowance for overhead charges has generally been substantially under-estimated by commissions and the Courts, as well as by engineers of limited practical experience in construction work."

The fact is, as before suggested, the omission of overhead charges and liberal allowances for omissions and contingencies by some American engineers, even by some who have obtained high rank in the Profession, has tended to bring the Profession into disrepute with the hard-headed business men and bankers of the country—the men who have been taught by sad experience. This is unfair to the Profession, as a whole, but it is not hard to understand. It is time that the members of the Profession recognized that an engineer, to be a safe adviser, must be a commercial engineer, and so recognize that his work must successfully stand the test of producing an adequate return on the investment. Many an engineer who has had wide experience as a constructor, but who has had little or no experience as an operator and administrator, woefully fails to meet his responsibilities in this regard.

Page 21.—"Preliminary Expenses."—The writer concurs most fully in all that the Committee says under this head.

Page 23.—"Incidental Expenses During Construction."—The writer concurs practically in all that the Committee says under this head.

The examples given in the tables are instructive and convincing; many equally convincing examples could be cited from the writer's experience.

Page 26.—The writer does not agree with the Committee in its statement, as follows:

"but, under a continuous system of regulation, engineering for additions and extensions, if charged to current expenses, should not be included in the valuation."

There are many conditions to be considered before such a sweeping proposition can be accepted: What has been the rate of return? What were the conditions before the commission control went into effect? What has been the system of accounting prescribed by the commission, and so forth? It by no means follows, because a system of accounting and returns has been prescribed by a commission, that the system

thus prescribed represents in every instance the best practice or even good practice; no, nor even the facts as correctly and fairly interpreted. Mr. Humphreys.

Page 28.—The writer's experience fully confirms the Committee's recommendation that contingencies should be covered by a liberal allowance and then "even under such circumstances the actual cost is as likely to exceed as to run below the estimate." It is fortunate for the Profession that the Committee has covered this subject of contingencies in such detail, and so convincingly. The writer wishes to emphasize certain of the statements, as follows:

"Large contingent expenses are necessarily incurred in practically all important public works." After naming many causes for contingent expenses, the report goes on to say:

"Such contingent expenses * * * with the exception of the last two [slipping of earth or rock, and making changes in plans which increase cost], should be included as contingent expenses in any valuation of an existing property because the existence of the property does not give any clew to the amount of most of the contingent expenses involved in its creation."

Page 29.—The writer also agrees with the Committee that, whether the owner actually paid for insurance protection or whether he assumed the risk himself, there should be added "the sum necessary to insure its whole value against fire." He would add, also, all other liability or risk incurred.

He also agrees that there should be included in the valuation an adequate allowance for property "destroyed, reconstructed, or not used because of faulty design or construction," provided this is not due to negligence. It cannot fairly be assumed that the human agencies employed are infallible. It can be assumed, only, that the human agencies are up to the average in this regard.

Page 30.—*"Development Expenses."*—These expenses should be included, as recommended.

Page 31.—*"Working Capital."*—There should be no question as to the correctness of the Committee's position, stated as follows:

"Working capital is essential to the proper operation of a public service property and should be included in the valuation upon which fair returns are to be based."

To those who have not had experience with public service commissions and the witnesses who appear before these commissions against the corporations, this statement by the Committee may appear to be quite unnecessary; but the statement is not only true but most opportune. The corporations not infrequently have to meet the claim that a public service corporation can operate with little or no working capital.

Page 32.—*"Depreciation."*—Although the writer is in accord with certain of the isolated statements in the report in regard to "depreci-

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ation", he is completely out of accord with the Committee in its final conclusions on this all-important subject.

In the first paragraph: In cases where conditions require an annual allowance for "depreciation" (and they are in the majority), the writer agrees with the Committee that there should be included in the rates not only a sufficient but a liberal allowance to cover this accruing loss. He does not, however, agree with the Committee in its use in this connection of the term "amortize." Going to derivation, it can be traced, in the Latin, to *ad*, to, and *mors*, death. We can refer to three definitions:

1.—"In English law, to alienate in mortmain, that is, to sell to a corporation."

2.—"To kill; to destroy."

3.—"To apply to a debt for the purpose of extinction, as by a sinking-fund."

Although in discussing the subject of depreciation, it may be allowable and convenient to use the term "amortize" by way of illustration, it cannot safely be used in an exact way to describe the working of a reserve or sinking-fund to provide against the non-uniformity of annual cost of renewals of plant. To "amortize" should convey the meaning that the investment is to be finally retired by the amortization; but such is wide of the facts, as the writer hopes to show. A municipality should amortize a bond issue which must be retired at a given date, and has been issued to finance a disappearing asset; or a capitalized leasehold; or capitalized standing timber; or mining properties. In all except the first of these cases the investment is day by day being extinguished or liquidated. In the case of a public service plant, day by day the several dates are approaching when its parts must be renewed so that the service may be continued. The investment is not extinguished, but is only given a different form.

Page 32.—Second Paragraph.—The writer completely disagrees with the Committee's statement, as follows:

"A depreciation allowance is, in effect, a payment from the ratepayer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

The report is not consistent on this point. For instance, on page 18 of the additional statement, it is said that "it would obviously be confiscatory if the value in such cases were to be depreciated." The cases referred to are the Replacement Method and the Sinking-Fund Method of computing a "depreciation" allowance. As thus computed, these are "depreciation" allowances, and therefore, according to the report, as above quoted, they should be regarded as a repayment of

investment "and should not appear in subsequent valuations of the property." Mr. Humphreys.

The Committee then goes on to say:

"It is necessary to adopt some practicable and equitable method of determining depreciation allowances which will meet the following requirements:

"1. The method should be one which will insure, with as much certainty as possible, payments to the corporation to amortize the various items of property when they cease to have value.

"2. It should so adjust the annual depreciation allowances as to distribute the burden as equally as practicable among the rate-payers of different years."

Referring to Requirement 1, the writer does not agree with the Committee, if the report means that the "payments to the corporation" amortize the investment.

Referring to Requirement 2, the writer agrees with the Committee, if the report means that the cost of all maintenance of plant should, by proper accountancy methods, be spread as uniformly as possible over the years during which the several parts of plant are to be operated in the service of the rate-payers; but it is difficult to see how this reading can find support in Columns 3 and 7 of the table on page 34.

In the writer's opinion, it is most unfortunate that the term "depreciation" has been used, as it has come to be used, to stand for the cost of deferred renewals of parts of plant as rendered necessary by physical decay (or wearing out to a point where no longer efficient and economical), or obsolescence, or inadequacy. If the subject is considered broadly, these deferred renewals, together with repairs and current renewals, may all be included under a single term, such as maintenance. Certainly we are not warranted in thinking that the money spent for repairs and current renewals is "in effect a payment from the rate-payer to the corporation of a part of its investment." If thus considered, then certainly it must be admitted that the money thus repaid to the corporation is at once re-invested by the corporation for the service-benefit of the rate-payer through the current expenditures for the up-keep of the property. If this is true of the repairs and current renewals, it is equally true of the deferred renewals, namely, "depreciation."

To make this point plainer, consider more in detail the reason for a "depreciation" allowance. Certain large payments have to be made from time to time for the renewal or replacement of certain more or less costly items of plant having a more or less extended life. If no thought were given to the subject, no provision as to accounting for, or financing, this loss would be made, and the cost of renewals or replacements probably could not be met from the income of the year when made, unless, possibly, return on capital were for the time defaulted. Furthermore, if in other years no provision were made in

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the Loss and Gain Account for this accruing liability, the yearly statements of profits would be inflated.

Now this, in a measure, is what did occur in many instances in past years, and particularly before the days of public service commissions. The conservative managers recognized these dangers, in a general way, and so they reserved part of their earnings, more particularly in good years, against the reduced earnings in "lean years." Certain of the ultra-conservative owners took advantage of good years to charge to loss and gain account and credit to plant account, or other asset accounts, large lump sums against "depreciation." The amounts thus written off were determined by the character of the year's business—extra fat or extra lean—and by the temperament of the controlling spirits in the board of directors, rather than by any attempt at accurate estimate or computation. Frequently, the very payments which had been anticipated in these periodic credits to plant account, namely, the expenditures for renewals or replacements, when actually made were debited again to loss and gain account, thus reducing the debit balance of plant account to a lower and lower percentage of the actual cost of plant. Where such credits had been made to plant account (or any property account) in anticipation of renewals later to be made, when those contemplated renewals were made, the expenditures therefor should have been charged to the plant account. The fact that, in some cases—perhaps too many cases—instead of this ultra-conservative practice, exactly the opposite course was followed, with the result that plant account became inflated, does not change the record as to the great majority of the public service corporations. Referring to the ultra-conservative practice, it is because so much of income was expended for extensions and improvements, which might have been legitimately distributed as return on investment, that the legitimate opportunities have been afforded for increasing the capitalization in connection with changes in ownership. It is also true that, in some cases, though in far less number than is popularly believed, this opportunity has been worked past the legitimate limit, resulting in "watered" securities. Many millions of dollars of "water" in securities have been eliminated by the investment of legitimate earnings otherwise applicable to dividends.

Many a public service corporation in the United States, if fairly valued, would now show a total far in excess of its total capital securities. Particularly is it true of many of our railroads, that they cannot earn 6% on a conservative valuation of their properties.

Now that the public service corporations are subject to regulation, especially as to rates, no such loose methods of accounting can safely be followed, and so plant account, or any property account, should not be written down on the basis of estimated future loss. Such losses should be covered by setting up reserve or liability accounts subject to

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correction to cover these estimated accruing losses. It should be understood that all expenditures to cover items which have been included in the estimate of accruing losses and contingencies, particularly renewals of plant, should be debited to the proper reserve or liability account. The writer's excuse for again emphasizing this point is the fact that not a few of those who discuss "depreciation" most glibly, assume that the corporations intend to debit these expenditures when made to loss and gain account and thus allow the reserve to go on accumulating indefinitely. If it is once understood why a "Depreciation Reserve" is established, it must follow that this is necessarily a credit and debit account.

If the debit to loss and gain account to cover estimated "depreciation" is made as a credit to a property account, instead of to a reserve account, we give, to those who are seeking an opportunity to confiscate investments, an opening to claim that debits to loss and gain, which in common prudence we have established to meet renewals and all contingent losses, are actual, present, and final lowerings of property values. Nothing could be farther from the true and legitimate theory of an annual allowance for accruing "depreciation," or still more, for contingencies.

Suppose, for instance, that we could in some way be insured against all yearly fluctuations in the cost of plant renewals, losses by uncollectable bills, liability for accidents, increase in cost of raw material, or labor, etc., then we could submit to a rate which contained no allowance for "depreciation" and contingencies, and there would be no opportunity for the Committee to report in favor of deducting accrued theoretical "depreciation" from plant valuation.

Now, in certain cases, and up to a certain point, such uniformity of annual charges for renewals can be found. In the case of a corporation having many plants, widely scattered, built at different times, the annual expenditures for renewals as a whole will tend toward uniformity. Here we have a case of averages. Certainly, in such a case, so far as the "depreciation" is covered by these annual expenditures, there should be no claim for allowance, and so no deduction from plant valuation because of theoretical accruing "depreciation;" and this because the accruing "depreciation", considered as a whole, is met by the annual expenditures for renewals. Why, then, should such deduction be made simply because these charges for renewals lack this uniformity of character? There is no fair reason, in the case of non-uniformity of annual expenditures for deferred renewals, why such deduction should be even temporarily made from investment, as shown in plant account, while the "depreciation" allowance by the Sinking-Fund Method is building itself up by its own interest accretions against the days of renewals. Naturally, under the conditions here assumed, analyses of the accounts and of the operations and estimates and com-

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putations would be necessary, to demonstrate the fact that the amount thus annually expended did equal the average cost of all renewals.

Where, in the writer's opinion, the Committee falls into error is in treating the "depreciation" allowance as if it were to pay for plant which is to go out of existence and hence the investment is to be extinguished, as far as the investor is concerned. This is quite at variance with the fact; for, though the particular part of plant which is being renewed or replaced does go out of existence, or is sold for service elsewhere, or as junk, there is another part of plant put in its place; and frequently a better part of plant, a part more up to date and hence more efficient and economical, and consequently more valuable from the standpoint both of the rate-payer and the corporation.

Page 34.—Having thus most briefly discussed the accountancy of "depreciation"—a phase of the subject to which, as it appears to the writer, the Committee has given insufficient attention—specific reference will be made to the Committee's "Equal-Annual-Payment Method of Computing Depreciation," as illustrated in the table under that heading and as discussed in the pages following.

Some of the explanations accompanying the table will be quoted:

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"The depreciation during each year, as given in Column 3 of the table, is exactly the same as if computed by the so-called Sinking-Fund Method; that is, it is the constant annual payment to a sinking fund added to the year's interest on the accumulated fund at the beginning of the year."

* * * * * * *

"Although the Sinking-Fund and Equal-Annual-Payment Methods, under the assumptions made, provide for the same annual rates, there is a difference in the principle involved. The Sinking-Fund Method provides for a uniform annual payment to the sinking fund which must be invested and earn interest in order that the accumulations may amount to the full value of the item of property at the end of its life, while the yearly depreciation allowances by the Equal-Annual-Payment Method include sums equivalent to the interest in the Sinking-Fund Method and therefore amount without interest to such full value."

These two statements must be considered together carefully. According to the report, these two methods are the same, and they are not the same. On the surface it would appear that the accumulation at any one time during the expected life would be the same; but if the equal-annual-payments are compounded in the same way as the sinking-fund payments, the total at the end of the 20 years will be found to be very different. In the case of the Equal-Annual-Payment Method, the total will be $152.83, instead of $100 by the Sinking-Fund Method. This is shown by the explanation of the Committee as quoted, for it is stated that the equal-annual-payment "is the constant annual pay-

ment to a sinking fund added to the year's interest on the accumulated fund at the beginning of each year." Mr. Humphreys.

The first of these two statements seems to warrant the assumption that the Committee claims that the "depreciation", as found at any time during the expected life by either the Sinking-Fund Method, as it really operates, or by the Equal-Annual-Payment Method, as illustrated in the table (and, therefore, excluding interest), is the accrued theoretical depreciation. In many other places in the report the Committee appears to support the same claim. This is not a fact, except at the very end of the expected life, and then only if the correct assumption has been made as to expected life. With the Sinking-Fund Method, at any time up to the end of the last year, necessarily the accumulations will not equal the estimated depreciation, for the very reason that the compounding of interest must be allowed to operate to the very end of the life to secure the necessary total accumulation. If the values are accepted as found in Column 2 of the table on page 34, "Value at end of year", at the end of 10 years the value of the plant, which cost new $100, is shown to be $61.96; whereas, by a uniform distribution of theoretical or "latent" depreciation, the value would be only $50. This shows that, if the Committee hopes to prove that its deduction for depreciation will always equal the accrued liability due to age, its own table proves to the contrary, and lays it open to criticism. (The writer will refer to this point later, in connection with remarks on commission and Court decisions.)

The only way in which such a claim could be supported would be by proving or assuming that "depreciation" increases year by year, as indicated in the table. Nothing of this kind can be proved. Quite apart from considerations of obsolescence and inadequacy, there is no law capable of general application by which one can measure accurately in advance the variations in the rate of actual depreciation of a plant composed of many parts varying widely in character.

If we confine ourselves to Columns 2, 3, 4, and 6, namely 5% sinking-fund computation and 5% return on investment, how do we find that this constant annual "Combined Depreciation and Return upon Investment" is secured? Simply by increasing arbitrarily the rate of depreciation allowance from $3.02 for the first year to $7.64 for the last year.

Of course, there is no such increase, in fact, in the depreciation in a well-managed public utility. Here, we must always remember, we are referring to the same item of property and are not considering extraordinary happenings or contingencies. If the latter are considered, no law can be established, and hence the rate assumed must be subject to modification, to meet changes in conditions as they develop.

Now consider for a moment the basis for using a sinking-fund rate to cover depreciation.

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The basis for this method is that if the part of plant is not to be renewed until 20 years have passed (to follow the Committee's table on page 34), then, though the loss has to be provided for annually in the rates, the actual payment is deferred, and so, if the *pro rata* amount is to be taken out of income each year, it can rightly be assumed that these advance payments should earn interest until the day they come due. If we place this money out at interest, an annual payment of 3.02% will accumulate to the required 100% at the end of 20 years, provided we can secure a compound interest rate of 5%; an assumption not warranted in practice.

Column 3 of the table, though footing up only $100, if compounded at 5%, as already stated, will produce a total of $152.83, which is the same as 20 payments of $7.64. It is to be noted that if we set aside $5 a year, the full amount required for the Straight-Line Method, and allow these amounts to compound at 5%, the sum will be $165.47, or only $12.64 more than the sum obtained by compounding the equal-annual-payment allowances.

If the 7% depreciation sinking-fund rate is used, as favored in Mr. Stearns' supplementary statement, the compounding of the depreciation allowances will produce $176 at the end of the 20 years, instead of the required $100. If the straight-line payments of $5 are compounded at 7%, they will produce $205.

In passing, it may be noted that, in Mr. Stearns' supplemental report, and in Mr. Snow's statement, in the case of a 7% return on investment, the annual combined payment is made to appear more like an "equal-annual-payment" by the assumption of a sinking-fund rate of 7 per cent. Of course, if the sinking-fund rate and rate of return are assumed as equal, this uniformity can be shown, provided the same arbitrary assumption is made as to the increase in the rate of depreciation year by year, as before discussed. The writer presumes it will be conceded that, in the operation of public utilities, the establishment of a sinking-fund outside the business is so rare as to be practically negligible when we are considering methods to be actually used.

The Sinking-Fund Method, however, cannot be ignored, because it does offer a process by which the annual loss due to depreciation can be provided for theoretically. The writer says "theoretically," when perhaps he should any "hypothetically", for no estimate on accruing "depreciation" can be depended on as to accuracy. Here is one of the dangers of the Committee's report, for the layman reading it might easily be led to believe that "depreciation" allowance is a quantity to be mathematically determined in advance. There is justification in making ample allowance for the safety of the undertaking, and this in the interest of rate-payer as well as owner. There is justification, also, in establishing a reserve against operating contingencies, outside of and beyond the factors usually included in a "depreciation" estimate. This

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reserve is subject to revision year by year by the commission in control, and if the reserve is not treated as a surplus, and is kept within reasonable limits, no injustice can result to the rate-payer.

If, then, we are placed in the position where theoretical or estimated depreciation can be met by a sinking-fund payment, there is an apparent reason why our annual allowance should be limited to that amount. We can claim a still larger amount if all contingencies are included; and here may be found a practical justification even for an allowance based on the Straight-Line Method.

In the table on page 34 the Committee has computed:

"the depreciation each year * * * by the so-called Sinking-Fund Method."

* * * * * * *

"Although the Sinking-Fund and Equal-Annual-Payment Methods, under the assumptions made, provide for the same annual rates, there is a difference in the principle involved * * * the yearly depreciation allowances by the Equal-Annual-Payment Method include sums equivalent to the interest in the Sinking-Fund Method and therefore amount without interest to such full value."

In other words, year by year the amount charged against the rate-payer increases from 3.02% until it is 7.64%, and, by reason of this increase, for which there is no warrant in actual conditions, the Committee is willing to deduct accrued depreciation from the reproduction-cost-new. The fact remains that the combined depreciation allowance and rate on the investment remains 8.02% each year, only if the required arbitrary assumptions are made as to the two rates being equal. With the assumptions made there is no difficulty in computing correctly the table on page 34.

If the same conditions are maintained for the 20 years, as shown for the first year of the table, the rate-payer pays 8.02% on the $100, and he pays no less when "depreciation" is deducted by the Committee's method.

It is proposed that we shall say:

"If you wish to deduct for depreciation in the valuation, you can do so provided you allow us to increase correspondingly year by year the allowance for depreciation."

For instance: For the twentieth year of our table we will accept a return of 5% on only $7.64 instead of on $100, or 0.38% instead of 5% on $100, provided you grant us an allowance for depreciation of $7.64 instead of the required $3.02.

But let us look a little further into this "constant each year" claim. On page 7 of the report, we read:

"Justice and equity require that in any regulation of rates fixing the amount which may be earned by a public service corporation there shall be taken into consideration two distinct features.

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"First, the annual return covering interest and profit, to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment."

In view of the happenings of the last few years, the writer presumes that the Committee would hardly claim that 5% is a fair return for "interest and profit * * * having in view the risks incidental to the investment."

Apparently, the Committee and the writer should be in accord in regarding 5% as inadequate to cover "interest and profit", for on page 36 we read:

"The foregoing statements [table on page 34] have been based on the use of 5% in all computations, but there are reasons, to be referred to subsequently, why the rate of return for the use of capital will generally be larger than the interest rate used in computing depreciation, and when there is such a difference in the rate, the combined annual depreciation and return for the use of capital, as given in Column 7 of the table, will not be strictly uniform from the beginning to the end of the twenty-year period."

The Committee would have been much nearer to the truth if the word "strictly" as qualifying the word "uniform" had been omitted.

Page 36.—Referring to the table on page 34, the report says:

"It has already been stated that in preparing the table only two of the three factors were taken into account, and the introduction of the third factor—the cost of repairs and operation—will affect the degree of uniformity because as a rule such cost increases as the property grows older. The inclusion of such cost, therefore, will increase the quantities in Columns 6 and 7 to a greater extent in the later than in the earlier years, making the results based on Column 6 somewhat greater in the later years than in the earlier years, and those based on Column 7, under most circumstances, more nearly uniform than they are in the table."

Referring to the table, it is found that the uniformity of the "Combined Depreciation and Return upon Investment", as given in Column 6, namely, $8.02 for each year, is sadly lacking in the quantities given in Column 7. In this last column the combined result is based on 5% sinking fund and 7% return on investment, instead of 5% and 5% for the uniform result of $8.02.

First, let it be noted that if this increase in "Cost of repairs and operation" operates to make the quantities in Column 7 more uniform, it operates to make the hitherto uniform results in Column 6 less uniform.

Referring to Column 7 (7% return) it is found that the combined payment for the first year is $10.02 and for the twentieth year is $8.17, a difference of 1.85 per cent.

Now, as to the compensating effect as claimed; namely, that "cost of repairs and operation * * * as a rule * * * increases as the property grows older."

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This depends on the design, construction, and operation of the plant, and particularly on the care and intelligence with which it is maintained and renewed. The result might easily be the opposite of that affirmed by the Committee; but here we must not forget the results shown in Column 3 of the table—"Depreciation during year". This is computed on the basis of a 5% sinking fund.

The results run from $3.02 for the first year to $7.64 for the twentieth year, or an increase of 150 per cent. There is no warrant in fact for any such increase in the "depreciation", but, such an increase having been claimed, there certainly remains no room for a further increase in "the cost of repairs and operation * * * as the property grows older."

Even as to "depreciation" (liability for deferred renewals), the rate at which this accrues is most problematical. Sometimes, and in some cases, this no doubt follows a straight line; sometimes it follows a most eccentric curve. If we attempt to base an estimate on the rate at which this item of loss actually accrues within the assumed life period, we shall involve ourselves in confusion worse confounded; particularly is this true if to the "depreciation", which includes obsolescence and inadequacy, we add all other contingencies. The most we can do is to make a conservative estimate as to the expected life of the many items of plant, so arrive at an average, and on these assumptions make a liberal allowance for the periodic renewals of plant and all contingencies. Having made this estimate as to the loss to accrue through a given number of years, conservative accounting and conservative financing suggest that this loss should be spread uniformly over the years, or *pro rata* on the present service and the prospective service, as estimated to the best of our ability.

Even though we could foretell exactly the rate from year to year at which this "depreciation" loss was to accrue, there would be no value in the information. Here the writer is not referring to the life of the parts, but to the rate at which the "depreciation" accrues within the assumed life limits.

If it were not for the practical need for spreading this loss *pro rata*, there would be no adequate argument to support a claim for annual allowance for "depreciation." This feature of the system of accounting (for that is what it is) may be vital for the protection of the investment, and particularly must be so where there are involved items of plant of large cost and long life.

One thing we can be sure of: these estimates never will be absolutely correct. In any specific case no final reliance should be placed on experience tables. These tables may be helpful in the hands of those qualified by personal experience to make the proper adjustments, taking into account design, construction, operation, methods of accounting, etc., and all through actual examination. The academic

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statistician, in the absence of personal needful experience, finds in these tables a fruitful source of information to be misapplied in developing any proposition he is for the moment supporting.

This question as to the propriety of introducing factor number three is suggested also by a statement on page 72 of the report, namely: "Depreciation allowances are credited to the capital account, and not distributed as dividends, and replacements become charges against capital." Here, again, how shall we draw the line between minor and major renewals or replacements?

On the other hand, if we charge to "repairs" all repairs and minor renewals paid for currently, and include in the estimate to cover "depreciation" only the major renewals not covered under "repairs", crediting the "depreciation allowance to a reserve account and charging to that account all expenditures contemplated in the estimate to cover "depreciation", there is thus presented the least opportunity for confusion or for unwarranted interference with the fixed capital accounts.

Again, referring to Column 5, the return on the undepreciated investment is found to be 7% for the first year and 0.53% for the last year. The total for the 20 years is 84.65%, or an average of 4.23 per cent.

Again, referring for support to page 7 of the report, even 7% might not be a fair return or an adequate return in many cases.

As the rate of return is increased, it becomes more and more apparent that the "Combined Depreciation and Return upon Investment", "will not be strictly uniform."

Referring to the table on page 34, and adding thereto to cover return on investment of 8%, 9% and 10%, it is found that the "Combined Depreciation and Return upon Investment" shows a variation (non-uniformity) between the first and twentieth years as given in Table 32.

TABLE 32.

Return Percentage.	Combined Depreciation and Return.		
	First year.	20th year.	Difference.
5	8.02	8.02	0.00
7	10.02	8.17	1.85
8	11.02	8.25	2.77
9	12.02	8.33	3.69
10	13.02	8.40	4.62

(Note.—Twentieth year, $7.64 \times 1.08 = 8.25$, $7.64 \times 1.09 = 8.33$, $7.64 \times 1.10 = 8.40$.)

Thus the table itself, when analyzed and expanded to cover other and legitimate rates of return, fails to support the report as to uniformity of annual payments. It is also found that, for the last year of the period, this Equal-Annual-Payment Method, for an increase

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of from 5 to 10% in the rate of return gives an increase of only (8.40 — 8.02) = 0.38% for "combined depreciation and return upon investment." Therefore it is found that this "uniformity" of the Equal-Annual-Payment Method depends for its "uniformity" on a rate of "annual return covering interest and profit" (page 7) which even the avowed enemies of the public utilities would hesitate to attempt to justify.

There is nothing to be found in the report, or in fact, which tends to make uniform either in rate or amount the annual expenditures for "repairs and operation" and deferred renewals. We can provide for uniformity of dividends by correct accounting methods, and by the building up of a reserve. As a general proposition, there is no way to make the total annual expenditures uniform.

If the Committee is looking for a constant return on the continuing investment, it has completely failed, as shown by Column 4 of the table on page 34.

But why involve "depreciation" with rate of return? There is no more reason why maintenance (of which "depreciation" is an element) should be thus involved in the effort to produce a constant quantity, than labor, coal, taxes, or any other expense item.

Here we are reminded that some statisticians and accountants differentiate between taxes and other items of expense; though taxes are no less an expense of operation because they go to the support of the several branches of government. Dr. Webber, the Chief Statistician of the New York Public Service Commission, First District, on a notable occasion deducted the amount paid out for taxes in connection with the computation of the cost per thousand for gas.

Consider for a moment the application of the Committee's method, as illustrated in the table on page 34, to the usual practice of investing "depreciation" allowance in plant extensions. If we invest $3.02 in our own business, that $3.02 thus invested each year must produce the 100% at the end of the 20 years. These extensions might be considered as carried into a development apart from the property as a whole. The earnings from these segregated investments would then have to produce sufficient gross earnings to pay for their own "depreciation" and also produce a net return which would meet the sinking-fund requirement of the original plant investment. If this method were applied to the "depreciation" reserve, as the property developed, the problem of keeping the earnings segregated so as to maintain on our books the many sinking funds thus required would present complications in the accounting department practically impossible to meet. Instead, therefore, of establishing these sinking funds on our books by segregating the earnings produced by the plant extensions paid for from the several "depreciation" allowances, the practical way is to consolidate all the earnings and compute the "depreciation" allow-

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ance on the total investment in depreciable property (or rather, such depreciable property as is included in our deferred renewal scheme) at the end of each year. This is not, as has been charged, asking for a return to the owners on the sinking fund as well as on the original investment, because we are computing the "Depreciation" allowance on the sinking-fund basis, and so the earnings from plant built from the "depreciation" fund should provide the interest necessary under the Sinking-Fund Method to produce the 100% required for renewals. This the owners must guarantee as trustees of the fund, if they invest in extensions. If the interest accumulations exceed the required amount, as should be the case, the owners are entitled to the excess because they have assumed the risk as guarantors.

On the other hand, plant extensions are not infrequently made in advance of a remunerative demand for service, and so, in this case, the return on these "depreciation" reserve investments may be less than the average rate of return on the investment as a whole.

The writer has shown that, by the Sinking-Fund Method, even when applied to investments in extensions of plant, 8.02% will cover "depreciation" and return on investment, if we assume 5% in each case. Now, the report specifically states that "depreciation" should not be deducted in the case of the Sinking-Fund Method. Mr. Stearns confirms this on page 18 of his supplementary statement, and says that thus to deduct from the value new "would obviously be confiscatory." Now, if the amount to be charged to the rate-payer in the two cases is exactly the same, why is it right in principle to deduct for depreciation in the one case and not in the other?

The writer quite understands that the Committee provides for deducting for "depreciation" by shifting the burden of the rate-payers from return on investment to "depreciation" allowance; but this is only an arbitrary rearrangement of the numbers between Columns 3 and 4. This increase in rate to be charged for "depreciation" allowance has no basis in the conditions met in practice. If the principle of the Sinking-Fund Method is appealed to, to obtain a basis for the first payment of $3.02, there is no reason why the same principle should not govern to the end of the assumed life of the plant.

Just so sure as engineers admit that "depreciation" (cost of deferred renewals) can be thus deducted from cost new, just so sure "depreciation" will be deducted irrespective of the methods of computation used.

No adroit tabulation of quantities based on arbitrary assumptions will serve to save us from the dangers to which this feature of the Committee's report must subject us.

As testified to by Commissioner Maltbie in the Bucyrus case, the New York Public Service Commission of the First District deducts for "depreciation" on the straight-line basis. Then, in a case before

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that Commission, for the tenth year, as shown by the table on page 34, 50% would be deducted instead of the 38.04% shown by the Committee in Column 2.

One thing cannot be lost sight of safely: this subject of "depreciation" is not easily understood. This fact is demonstrated by this report and the discussions thereon; and this quite apart from the question: who is right and who is wrong?

This being the case with the engineers and operators of experience, we certainly cannot expect the commissions and Courts to understand the subject readily, even though all the commissioners and all the judges may be keen to learn the truth through the desire to hold the scales of justice evenly.

Experience, however, does not teach us that all these officials are keen to do exact and even justice. On the contrary, acting under the influence of their advisers, many of whom treat this subject most academically; they have already in not a few cases shown a marked disposition to decide against the investor when a plausible reason could be found for doing so.

The commission and Court decisions on this question are found to be conflicting when carefully analyzed; and this is easily understood when we see how inadequately the subject of depreciation has generally been presented by the witnesses.

It is, then, our duty, as men charged with a very special and grave responsibility, to be prepared to place before the commissions and Courts this subject of "depreciation" and all involved questions in the simplest possible form.

Here the writer may appropriately quote from an address delivered in March, 1914, before the American Railway Engineers Association of Chicago by Judge C. A. Prouty, formerly of the Interstate Commerce Commission and now Director of Railway Valuations:

"In the last fifteen years the price of every article of universal and general consumption, with one exception, has advanced, and that exception is transportation by rail. If transportation by rail alone has not advanced it is because railway engineers, by providing a better way and a better machine to operate upon that way, have rendered it possible to make economies which have offset the increased cost of labor and of supplies. The United States is about to enter upon a work of very great consequence, and the railway engineers are identified with that work. The problem is not altogether an engineering problem. It smacks of economics and of law and of politics, but the business of ascertaining and collecting a great part of the information is peculiarly a task of the engineer's, and I am not certain but the engineer may finally turn out to be the best qualified of all to consider the other problems.

"Now, when I began this work I was told that an engineer could measure and figure with accuracy, but that he was not competent to discuss the intricate problems presented by the valuation of our rail-

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roads, and that we must be very careful not to let him undertake to discuss them. I am fast coming to the conclusion that he may be better qualified than anybody else to discuss them. I have been a lawyer all my life and I believe that my profession is the greatest of all professions, but I am obliged to admit, and I think I have said this before, that the training of a lawyer does not fit him for the discussion of questions of this kind. A lawyer by his training is a partisan. If you go back to the law school you will find that he is given a certain thing to affirm, and it is his business to make out that case. In life he has one side of a controversy, and he must establish that his side is right.

"The engineer is entirely different. He begins in school with mathematics. There is but one answer to a problem in mathematics. It may be hard work to find it, but there never can be two answers. When the engineer gets into life it is still his business to find the fact, and just according as he can find the fact and be certain of the fact, he grows. If an engineer designs a bridge, and that bridge falls, his reputation falls with it. He cannot call in the undertaker and bury his blunder, as the doctor does. He can't charge it to the stupidity of a court, as the lawyer does. He must assume the burden of it himself. So I say that the very training of the engineer fits him for that precise kind of thinking which is essential in the working out of these new problems where absolute impartiality is necessary."

Here at the outset it should be kept constantly in mind that the annual allowance for "depreciation" is not susceptible of being determined with mathematical accuracy. This, it is presumed, every member of the Society, who has had a long and wide operating experience and has profited thereby, is willing to concede.

The depreciation allowance is based on an assumed life for each part. Referring to the table on page 34, let us suppose the life of the part of plant under consideration, or the time for renewal, extends to 21 years. This would be an unusually close approach to the estimate. Then, as at the end of the twentieth year the total cost has been deducted through the depreciation allowance, for the twenty-first year there will remain no investment on which to claim a return, and no ground for claiming a depreciation allowance. Hence, there will be no ground for claiming that the service rate shall include a return on the investment and a depreciation allowance; and this condition would continue until the plant was actually renewed.

This indicates that the so-called Equal-Annual-Payment Method is bound rigidly to its expectation-of-life estimate; whereas, the Sinking-Fund and Straight-Line estimates as generally made are subject to correction year by year.

What is plain to the engineer, however, may not be plain to the layman. May the writer repeat the suggestion, that the laymen who read this report will almost to a man receive the impression that the allowance for "depreciation" is a quantity which can be determined

accurately in advance of the established facts, and that the Committee favors deducting from "depreciation". Mr. Humphreys.

Even in a case where the layman might be inclined to think that a readjustment was in order, it should still be borne in mind that an apparent excess in the reserve might be justified by the chances, always more or less probable, of extraordinary happenings beyond the power of mortal to foresee or provide against, except through such a reserve. Those, at least, who have had any wide experience in the operation of public utilities must be in a position to know that such operating contingencies (quite apart from the contingencies connected with the construction period) cannot be disregarded safely.

Page 39.—"Straight-Line Method."—The report states that this method "assumes that the value of the item of property under consideration decreases an equal amount in each year of its life" and therefore (page 40) "it is inequitable as it makes a much greater demand on the rate-payers of the earlier years than on those of later years."

The writer is unable to agree with the Committee as to either of these statements.

Neither the Straight-Line Method nor the Sinking-Fund Method necessarily assumes anything as to the actual rate of depreciation, but, as a scheme of accounting, each of these methods endeavors to place an equal annual burden on the years of the expected or assumed life.

Instead of placing an equal annual burden on the years the burden could be spread *pro rata* upon the sales of service; this would require an assumption as to any expected change in the annual sales of service. This change in method of distributing the burden has no bearing on the question of deducting for "depreciation."

If the cost of deferred renewals ("depreciation") is treated as an item of cost or expense, it is at once seen that either the Straight-Line Method or the Sinking-Fund Method is more "uniform" as to burden than the Committee's "Equal-Annual-Payment" Method.

As before stated, there is no more reason why the deferred or periodic renewals ("depreciation") should not be considered an item of operating cost than that repairs and current renewals should not be thus considered. Broadly considered, repairs and all renewals are included in the cost of maintenance. In fact, not a few accountants cover all maintenance as above described by an estimated annual allowance, carrying this periodically into a Reserve Account as a total and then debiting against the credit balance thus created the actual payments for repairs and current renewals as well as deferred or periodic renewals. The intention or hope is that by thus combining in one estimate repairs and all renewals the annual charge for all maintenance can be kept more nearly uniform.

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Under such a system as this, how would the Committee support its claim for deduction of depreciation? If part of the credit balance of this inclusive reserve happened to be applicable to repairs and minor renewals, would the Committee also deduct therefor?

If so, to be consistent, they should also deduct for repairs and minor renewals as currently debited direct to loss and gain where the total is found to be below the annual average—namely, indicating an accrued liability for repairs.

The writer is willing to admit that the Straight-Line Method is open to criticism, but not for the reasons given in the report. This method calls for the advance payment of the loss due to prospective renewals, whereas it can be claimed on behalf of the rate-payer that these annual allowances should be permitted to earn interest until the days come to pay for the renewals, thus reducing the allowance to an annuity basis. Even then, however, the Straight-Line Method is not as unfair as it may at first seem. It is to the interest of the rate-payer as well as the owner that all accruing losses and all contingencies should be provided for liberally. On page 70 of the report we find a statement which appears to support this position, as follows: "A good sized surplus is desirable for preventing frequent changes in rates and dividends, and for protecting stable conditions." The writer prefers to regard this accumulation as a reserve rather than a surplus, especially as being fairer to the rate-payer. Governmental control of gas companies in Great Britain, without directly authorizing an allowance for "depreciation", thus provides through insurance reserves for the distributing of all accruing losses (including "depreciation") over the years, and also provides for changes in prices of supplies and materials and rates of wages, thus insuring stability of dividend rates and so making these public utility securities more desirable for trust investments. Evidently, the true significance of "depreciation" is better understood by the authorities in Great Britain than by those in the United States.

If by the Straight-Line or any other method, the reserve is found to be accumulating in excess of the apparent need, all things considered, including stability, the scheme is open to revision.

And this is where the Commissions should exercise their powers, always provided they are sure of the ground; but, commission or no commission, no estimate to cover accruing "depreciation" should be allowed to stand year after year without question. Such an estimate should always be open to revision in the light of changed conditions and increased experience. Nothing, to the writer's mind, is less defensible than the setting up of a "depreciation" scheme by rules and tables, to be from then on regarded as sacrosanct.

Before we can decide that the reserve is too large, we must, however, take into account contingencies, and not govern ourselves entirely by

the things in view or even in mind. Here is another reason why a straight-line allowance should not be condemned hastily. Mr. Humphreys.

Let us for a moment consider what would be the result of a straight-line allowance, provided the estimates were by chance found to be actually accurate. Let us see what the Straight-Line Method would provide in the way of return of the investment to the owner in addition to providing for "depreciation."

Referring to the tables on pages 34 and 40, the straight-line annual allowance would be $100 \div 20 = 5$ per cent. The amount required by the Sinking-Fund Method would be 3.02 per cent. The difference or excess is 1.98 per cent. This 1.98% also might be allowed to accumulate at compound interest at 5 per cent. At the end of 20 years this would amount to $65.47.

Thus, by the "Straight-Line" Method, assuming that the life expectation had not been over-estimated, and assuming a return on the full 100% of the investment each year (as differentiated from the Equal-Annual-Payment Method), after providing for deferred renewals, $65.47 of the investment would be returned to the owners, but with no reserve remaining for contingent happenings.

Page 41.—"Actual Inspection Method."—The writer is unable to find any ground for the following statement in the report:

"This method does not take account, to any considerable extent, of functional depreciation, and the great injustice which may be done a corporation by the failure to take such depreciation into account has already been stated."

It is because the writer wants to provide as completely as possible against the "great injustice" referred to that he here differs from the report.

There is no reason why the valuation of property by actual inspection should not take full account of all the elements included in "depreciation." Through actual inspection by competent experts, the probably remaining useful life, having in mind physical decay, obsolescence, and inadequacy, can be determined much more accurately than by reference to tables as used in the making of an estimate perhaps years before the making of the appraisal. Backsight is certainly more to be relied on than foresight. An appraisal without competent inspection would be valueless.

The report itself says that such actual inspection should be made (page 42) "partly for the purpose of determining the probable future life of the various items of property."

Page 42.—"Replacement Method."—There is no good reason why this method, in certain cases, properly applied, should "hold back from the corporation an annual sum for depreciation."

If the current annual expenditures for replacements do not provide for the average annual cost of such renewals, including all elements of

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"depreciation" (and, as a rule, they do not so provide), the corporation is entitled to an allowance therefor to be included in its total of annual cost of service; and this, as before stated, is simply to spread this loss or cost item as uniformly as possible over the years of service. The best way to insure the granting of such allowance is to educate the commissions and Courts as to the real significance of "depreciation."

Page 42.—"Application of Equal-Annual-Payment Method."—As before suggested, in establishing a scheme on which to base an annual allowance for "depreciation", the line then must be drawn between the classes of items which are to be charged directly to operating cost through "repairs" or some equivalent account, and the classes of items which by journal entries are to be debited to operating cost and credited to reserve. When these latter items of deferred renewal are actually paid for, the debit, of course, is to be made against this reserve. Thus the balance is maintained and we are constantly afforded opportunities to check up our estimates on "depreciation" through the facts as they develop. By crediting "depreciation" allowance direct to plant account, no such segregation of the items going to make up the average annual cost of renewals is available.

The last paragraph of this section is as follows:

"Under a continuous system of accounting, small items of property purchased and charged directly to current expenses should obviously not be included in the valuation of the property upon which the annual return is to be based."

There are many reasons why this is not "obvious". How are these items to be located with any degree of exactness?

Suppose, in the estimate for "depreciation" allowance, it is assumed that such "small items of property" are to be "charged directly to current expense" so as in part to compensate for the wasting of assets, or, to keep the assets at par? This is not an exact method, nor is any method used for covering "depreciation" mathematically accurate. Just so surely as dependence is placed on any method as being mathematically accurate, just so surely will we go wrong. We can keep nearest to the truth only by constant watchfulness and the constant exercise of common-sense methods, including full co-operation between the several departments of design, construction, operation and accounting. After which there should be full and frank co-operation between the corporation and the commission.

Page 44.—"Decisions of Courts and Commissions in Regard to Depreciation."—Commission and Court decisions, as affecting depreciation, cannot be followed blindly, because, as has been well said, "the subject of depreciation as affecting values has never been presented adequately to any rate-fixing tribunal." The writer would enlarge on this by saying that one reason why the presentations have not been adequate is because of the lack of adequate preparation and training on the

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part of commissioners and judges, and so they have not been able to understand the presentations as made; and, again, we may be reminded that these decisions are not in agreement with each other. Then how shall we choose?

The Minnesota Rate Case is much quoted, and will largely influence the determination of the questions under discussion until reversed or amended by later decisions. In the meantime that decision should not necessarily control our judgment. With due respect to all concerned in this case, the questions involved in "depreciation" were not adequately presented. Furthermore, this case does not sustain deduction for accrued "depreciation" due to expired life. Justice Hughes does not define what or how much should be deducted. It is quite possible that he may have agreed with the New Jersey Commission and with Judge Hough in the Consolidated Gas Case (New York) to the effect that the deduction, if any, should be confined to such an amount, generally comparatively insignificant, as would re-establish the property in a thoroughly efficient present condition from the operating standpoint. This is a very different proposition from deducting for accrued estimated "depreciation" due to aging.

As stated by H. P. Gillette, M. Am. Soc. C. E., in his discussion of the paper by W. J. Wilgus, M. Am. Soc. C. E., on "Physical Valuation of Railroads",* referring to the Minnesota case:

"The inconsistent use of words and lack of proper definitions of terms account as much as anything for this erroneous ruling, for the Court was attempting to use symbols which it did not fully understand, and did not understand because they were not clearly and properly defined by any witness."

It is not for us to follow meekly any decision which, from our special training or experience, we believe to be in error. For instance, on page 46 the report quotes an opinion by Judge Gray in which he states that while the majority opinion of the Court is in favor of "straight-line" computation of "depreciation", he is "personally of the opinion that the creation of * * * an amortization fund furnishes the best rule for adoption." And it is this last rule which has more generally been followed by Courts and commissions.

Judge Haight's concurring opinion gives his reasons for accepting the "Straight-Line" Method and for rejecting the sinking-fund plan, which reasons are by no means convincing. For instance, Judge Haight says:

"Suppose the average life of the tangible property of a railroad, outside of the land itself, to be sixty years and the cost of reconstruction to be sixty million dollars, it would follow that one million dollars would have to be used each year in reconstruction, and that amount would have to be annually used for that purpose; but under the plan

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 265.

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adopted in this case, instead of deducting from the gross earnings the amount necessarily expended for that purpose, a small fraction of that sum, viz., $4 200, only is allowed to be deducted, a sum which, with interest compounded for the next 60 years, would amount to a million dollars. Under such a plan the company would be practically prohibited from annually constructing a portion of its road and thus prevented from keeping it in that state of efficiency which the public demands."

This quotation shows conclusively that Judge Haight did not understand the Sinking-Fund Method, as applied to meeting "depreciation"; and also that he did not understand that the assumed life of a plant in connection with the "depreciation" problem means the average life of the plant, as made up from the assumed life of the many parts of plant, these parts of plant to be renewed at different times widely apart, and parts of the plant to be renewed a number of times before the end of the 60 years. The sinking fund, in fact, is thus seen to be the sum of many sinking funds, each fund being designed to provide for the renewal of its life-group. And so, the sinking fund, being made up to meet the renewal of each life-group of plant, the money will be found in the fund as required; or, rather, would so be found if the life of each part as carried into the average were found to be as assumed.

Furthermore, evidently, Judge Haight failed to see that this "depreciation" is the accruing loss over and above the current expenditures for repairs and minor renewals; in other words, the current up-keep or maintenance. Still further, there is no warrant for contrasting the expenditure of $1 000 000 a year on the straight-line plan, with $4 200 a year, on the sinking-fund plan. As Judge Haight states, the $4 200 a year would produce at compound interest (at 4%, though not so stated) only $1 000 000 at the end of 60 years; whereas, the fund would have to produce at the end of the 60 years, $60 000 000; and for this the annual sinking-fund payment required would be $252 000 instead of $4 200.

The Committee (page 40) has declared that the "Straight-Line" Method, as approved by the New York Court of Appeals, is "inequitable" and "permits the corporation to earn for depreciation in the earlier years a larger sum than is necessary to maintain the full value of its investment."

Although, as before explained, the writer differs from the Committee in believing that the "Straight-Line" Method is necessarily inequitable, if engineers can agree on one point here suggested, in addition to the other points open for agreement as set forth in the report, a great gain will have been made, namely: that commission and Court decisions and opinions are not to be accepted blindly by those who have the right to believe they are by training and experience better informed on certain of the elements involved than are the commissioners and judges. (See Judge Prouty's statement previously quoted.)

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Much of the confusion of thought on this vexed question of "depreciation" comes, as it seems to the writer, from the fear on the part of the corporation representatives, and the rather general opinion on the part of commissioners and judges, that if a deduction for "depreciation" is not made in valuations of property, then the right to demand an annual allowance for "depreciation" (deferred renewals) is surrendered. This error would be less in evidence if we stopped talking about "depreciation" and turned our thoughts to deferred or periodic renewals.

To render adequate service, the whole plant must be kept in continuous service, and at the same time money must be earned to renew the plant as the deferred renewal dates mature. In the meantime, it is not 90%, or 80%, or 70%, or 60%, or 5% of the plant which is rendering service; and the test to be met by the corporation, as far as the purchaser of service and the commission are concerned, is the adequacy of that service.

Unfortunately, this test is ignored by some of those who influence strongly the views of the commissioners and judges.

The following brief quotation is from a paper read before the convention of Railway Commissioners in October, 1913, entitled, "Depreciation and its Relation to Fair Value", by Dr. A. F. Webber, Chief Statistician of the New York Public Service Commission, First District, to whom the writer has previously referred:

"When that contention is made [not to deduct for estimated 'depreciation'], I always feel like asking whether a grindstone that is owned by a traveling scissors-grinder is worth 100 per cent. of its value after it has been half worn off. Of course, it may do its work just as efficiently as a new grindstone, but certainly the peddler could not obtain 100% of the value—and by that I mean the cost—after the grindstone has been half worn out."

The writer's reply to this is, that the grindstone would not bring as much as a new one if the scissors-grinder should attempt to sell it; but, if he is to stay in the business, he will continue to use the grindstone until he has to buy a new one because the present one has become inefficient; and in the meantime he must provide for the cost of the new grindstone in his charges for grinding. This he can do without change of rate due to the aging of the grindstone, for, as Dr. Webber says, "Of course, it may do its work just as efficiently as a new grindstone." Those who entrust their scissors to this man do not know and do not care how much or how little the grindstone has been reduced from its original diameter. They do care, though, as to how well the scissors have been ground. The scissors-grinder is not selling grindstones; he is selling service.

The writer will quote from this paper once more:

"In a memorandum on 'Depreciation In Its Relation To Value' recently submitted to a commission by the attorney for a lighting com-

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pany involved in a rate case, he declared that the opinion of the United States Supreme Court in the leading case of Knoxville *vs.* Knoxville Water Co. should not be interpreted to the effect that 'accrued depreciation must be deducted from reproduction cost new in order to ascertain the fair present value of depreciable tangibles', and then went on to say that 'The subject of depreciation as affecting values has never been presented adequately to any rate-fixing tribunal. As a consequence there have been conclusions which are absurd. To use a homely illustration, it is as if it had been determined that the price which a farmer is entitled to receive for his milk should be measured by the unexpired, normal, useful life of the cow from which it is drawn'.

"The illustration is a most unfortunate one for the argument against the deduction for depreciation in determining fair value, because age is a very material factor in the price or selling value of a cow. Nowadays, when a milch cow is kept in the dairy only four or five years, a farmer who could be induced to give up as much money for a six-year-old cow as for a two-year heifer would not continue long in business unless he had some property still unencumbered on which he could borrow funds. The mere fact that the attorneys who argue public utility rate cases are usually city men who have forgotten all they ever knew about cows and horses seems hardly sufficient to explain their attitude, because the automobile that carries them on their daily round of business and pleasure is a standing protest against their ingenious sophistries. I have yet to meet a lawyer who is willing to pay as much money for a used car as for a new car!"

The writer agrees with Dr. Webber that a "most unfortunate argument" has been presented, but is afraid that he does not agree as to the person responsible for the "unfortunate argument." As he is not the owner of an automobile, but is the owner of a considerable number of horses and cows, perhaps he is able to face Dr. Webber's bucolic sarcasm.

The attorney quoted was not talking about the price or value of cows, but about the price or value of milk. The farmer must get such a price for his milk that he will have in reserve enough to buy a new cow (or shall we say heifer) when the present cow permanently dries up or is not longer profitable or safe as a milker, and in the meantime he will go on charging the same price for his milk—that is, for his service to the milk buyer. He will not charge a graded price for the milk according to the age of the cow which produces it.

Page 47.—"A Fallacy in the Discussion of Depreciation."—The report says:

"At the risk of repeating what has already been stated, the Committee will refer again to the fallacy of the view expressed by engineers of much experience in valuation, that rates based upon the depreciated value of a property are necessarily lower than those based upon its full value."

Those who disagree with the Committee on this subject of "depreciation" might in turn claim that the Committee is advancing the

"fallacy". The Committee quotes from the writer's discussion on the paper, "Physical Valuation of Railroads," by William J. Wilgus, M. Am. Soc. C. E. (page 49), as follows: Mr. Humphreys.

"'In the case of a public utility, adequately maintained and rendering efficient service to the public, to deduct for accrued depreciation according to the practice of some of our theorists, necessarily results in confiscation of investment.

"'Certain appraisers of reputation have taken the ground that if, for instance, railroad ties have an effective life of 10 years, and one-tenth of the total number is renewed each year, there is no need for a depreciation reserve, but the value of these ties as a whole should be depreciated 50 per cent. Nothing, according to the writer's way of thinking, could be more fallacious. The property, as far as this item is concerned, is thus maintained at maximum efficiency. What more could be done? Why then should one-half of this portion of the investment be confiscated?'

"It will be observed that the writers in all these cases ignore the fact that a depreciation allowance earned from the rate-payers is a return of part of the investment, and that after that part of the investment is returned, it is neither inequitable nor confiscatory to base future returns upon the depreciated value, which is the remaining investment in the property. They also erroneously assume that the rates necessarily decrease when the depreciated value is used."

In this the writer is charging the other side with dealing in fallacy; and, as in the case of the Committee's charge, it is simply an assertion, unless the writer can support his contention.

It is true that he ignores "the fact that a depreciation allowance earned from the rate-payers is a return of a part of the investment", and his reason for doing so is because that is not the fact. It is not a return of investment, but an item of cost, quite as much as repairs and current minor renewals. The fact that this allowance covers major and deferred renewals does not make it any more a return of investment than the amount taken from rate-payers to pay currently for minor renewals as included in repair account.

Again, the writer does assume, but not erroneously, "that the rates * * * decrease when the depreciated value is used."

He does not assume that it is so "necessarily", but he does assert that this is what does happen under commission control, and has so happened in cases in which he has been directly concerned. If the valuation is made a basis for rate-fixing, and that valuation is reduced for any reason, deduction for "depreciation" or otherwise, so far as the valuation affects the rate fixed, the "depreciation" deduction, if honestly made, must operate against the owners.

After asserting that this assumption as to the effect of "depreciation" deduction is erroneous, the report goes on:

"It is true that of the return on a given item of property, the part applicable to dividends becomes less as the property grows older,

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but this is as it should be; a part of the capital invested in that item has been returned and has either been invested in additions or replacements, or temporarily in a depreciation fund, so that the whole of the original investment is at all times earning returns applicable to dividends."

The writer has previously suggested that this report will give those opposed to public utilities the opportunity to claim that this Special Committee of the American Society of Civil Engineers favors deducting for accrued "depreciation"; and this without regard to the Equal-Annual-Payment Method as advocated by the Committee. In relation to this matter it is particularly to be noted how effectively this last quotation from the report could be used against public utilities, especially when quoted out of its context.

The Committee states first that it is an error to suppose that "the rates necessarily decrease when the depreciated value is used", and then goes on to say that "of the return on a given item of property, the part applicable to dividends becomes less as the property grows older, but this is as it should be." Apparently, the Committee thinks that the commissions treat public utilities more gently and generously than they have the right to demand—a tendency the writer has not been fortunate enough to experience.

Again, the Committee states, "the whole of the original investment is at all times earning returns applicable to dividends."

May the writer suggest that this shows that his charge of fallacy can be maintained, and the Committee's charge of fallacy must fall?

The Committee states that with the Sinking-Fund Method "depreciation" should not be deducted. By the Sinking-Fund Method, whether the allowances are invested inside or outside of the business, the "depreciation" fund thus invested is not primarily earning dividends, but is earning the difference between the sum of annual payments and the required 100 per cent. When the writer says "depreciation" should not be deducted, he means the "depreciation" estimated by the Sinking-Fund Method.

If the owners take the risk of investing this trust fund in extensions of plant, they should be reasonably sure of earning something in excess of the required sinking-fund accumulations, for otherwise they would not be warranted in thus investing, but should establish a sinking fund outside the business, and thus relieve themselves of responsibility as guarantors.

The "investments in additions and replacements" are made from the allowances for "depreciation" (and these allowances are less than the Equal-Annual-Payment allowances), and these payments, so far as they go into "replacements", go where they are intended ultimately to go, and in the meantime such amounts as go into "additions" are

invested in accordance with the interest compounding requirement of the sinking-fund.

The writer has already shown in detail why it is that, by the Equal-Annual-Payment Method, the pretence can be made of deducting for "depreciation." He has also drawn attention to the fact that the Committee unqualifiedly condemns the deducting of "depreciation" when computed by the Sinking-Fund Method. Then, as one of "the writers in all these cases", he advocates what the Committee approves, for his advocacy was based on the Sinking-Fund Method of computation.

Page 49.—"Past Practice has Underestimated Depreciation."—The writer agrees that this has frequently been the case. It is not true as to all cases. In many cases "depreciation" has been allowed for too generously, and in most of such cases the "writing off" for "depreciation" has carried the book value of property far below its present value, thus, under present conditions, working an injustice to the owners.

Page 50.—"Appreciation."—The following is quoted from the report (page 51):

"Attention is called to a very clear discussion of this question by Commissioner Maltbie of the New York Public Service Commission for the First District, *in re* Gas and Electric Rates of the Queens Borough Gas and Electric Company."

Then Commissioner Maltbie is quoted as follows:

"Thus, land has been taken at its fair value and not at its original cost, and the annual appreciation of land has been treated as a profit. By this method, all property is treated absolutely alike, as Judge Hough suggests."

Here Mr. Maltbie has apparently cited Judge Hough as supporting his revolutionary proposition; but this is not warranted by the facts. Judge Hough's statement referred to the capital value of the assets of the Consolidated Gas Company. He took the position that, in applying cost of reproduction, depreciation and appreciation must be applied impartially, whether this course resulted, in individual cases, in an increase or a decrease from the original cost of production.

In treating income and expense, he did not add increase of real estate value.

Furthermore, as to "depreciation," he only deducted a small amount, comparatively, to put the plan in "an operating condition" as good as new—the amount deducted was only $604 000 from nearly $50 000 000 of tangible assets.

In the writer's opinion, it is most unfortunate that a Committee composed of such prominent members of the influential American Society of Civil Engineers should be willing to endorse this opinion of Commissioner Maltbie.

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To take an extreme case to show the principle: if appreciation of land value alone should be sufficient in amount to give a fair profit per unit of service and then be "treated as a profit", no profit from sale of service would be required, and so the rate could be fixed at cost to deliver. Thus the owners would be obliged to accept an estimated prospective profit on their land, with this prospective profit locked up in the land occupied by its plant, and thus there would be no cash profits to cover dividends. Would the commission authorize borrowing money on these hypothetical land appreciations for the payment of dividends? Or would the owners be satisfied, in the absence of dividends, to have placed before them certain journal entries covering hypothetical book profits.

It is hardly reassuring to note that, in the opinion of Commissioner Maltbie, all this depends on "only a simple mathematical calculation." Mathematical calculations are interesting, and they form the stock-in-trade of our present-day statisticians; but they do not pay dividends, nor even buy bread and butter.

Evidently, the Committee when it wrote its report was not aware that the Appellate Branch of the New York Supreme Court, in the Kings County Case reversed Commissioner Maltbie on his unique proposition to transmute appreciation in land values into income. The Court said:

"So long as the land is held and used for such purpose increase in value cannot be considered as income and available for the payment of debts, taxes, or dividends."

Page 70.—"Summary."—Having been forced to disagree with the Committee on certain points, and emphatically as to deduction of estimated "depreciation", from valuation, the writer is more than pleased to find that he is in agreement with the Committee as to many of the Sections under the head of "Summary." He is in agreement as to Sections 1, 5, 6, 8, 9, 10, 12, 13, 15, and 23.

Section 2.—The writer is able to endorse this section if there be inserted between the words, "for" and "all", in the fourth line from the end of the paragraph, the words, "the renewals of" and if there be substituted for the words, "cease to have value", in the next line below, the words, "are displaced."

Sections 3 and 4.—The writer disagrees as to depreciation, for the reasons stated.

Section 7.—The Appellate Branch of the Supreme Court, New York, recently reversed the ruling of the New York Public Service Commission in the Kings County Lighting Case, previously referred to, that the cost of paving laid subsequent to the laying of mains should not be included in valuation. This decision has in turn been reversed by the Court of Appeals, again demonstrating that Court decisions and opinions contain no attribute of infallibility.

Section 11.—"Going Value" may be in excess of cost of producing that value, though this excess may be difficult to measure. It is not necessarily true that "development expenses connected with ordinary additions to plant are a much smaller percentage than those connected with the original plant." The development of new territory, perhaps made obligatory by commercial conditions or commission's orders, may at first be a most unremunerative undertaking. Mr. Humphreys.

Section 14.—The writer disagrees as to depreciation deduction, as explained.

Section 16.—The writer entirely disagrees as to Equal-Annual-Payment Method, and again asserts that, under reasonable conditions, there is no warrant for the statement that these combined amounts "are substantially equal in each year of the life of such item." He also holds that there is no more reason for deducting "depreciation" by this method than by the Sinking-Fund Method, except in the manipulation of figures, which transfers the decreasing return on the investment to the increasing charge for depreciation.

Section 17.—As already explained, the writer does not agree with the Committee's statements or conclusions as to the Sinking-Fund Method.

Section 18.—There is no reason why the "Actual Inspection Method" cannot be made to operate fairly to corporation and rate-payer. In fact, inspection is necessary, and no proper valuation can be made without inspection.

Section 19.—To treat appreciation in the value of occupied land as income, instead of as increase of asset, is unworkable, indefensible, and absurd.

Section 20.—There is no objection to paying proper regard to the Minnesota Rate Case, but that decision is not necessarily to be followed for general application, and is a good subject for amendment in certain particulars.

Section 21.—Deficiency in past earnings may properly be taken into account in determining whether the replacement of plant rendered obsolete by sudden and unexpected changes in the art has been provided for in the income from rate-payers.

Section 22.—If the value can be found in the property, whether produced by cash expenditures or by fortunate circumstances united with foresight, the property as a whole belongs to the corporation.

Although the writer has taken much space to express his views on the questions treated, it is not necessary to state that the opportunities for misunderstanding are so many, and the questions are so momentous, that all may feel justified in making every possible effort, including repetition and paraphrasing, to reduce the chance of misunderstanding to a minimum.

J. H. H. Muirhead, M. Am. Soc. C. E.—Noting some confusion as to the meaning of certain terms, the speaker suggests that the Com- Mr. Muirhead.

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mittee include in the report some decided and definite meanings of the technical terms. One word has been used by one member to mean a certain thing and by another to mean something entirely different. At the same time some members have offered explanations of a modern nature for some terms, and therefore far in advance of the various dictionaries. Something of this nature should accompany the final report.

Public utilities come under three headings. First.—Utilities such as railroads, telegraphs, telephones, etc., of an interstate nature owned and operated by corporations the stocks and bonds of which are owned by the general public. Second.—Public utilities owned and operated by municipalities. Third.—Public utilities (like the New York subway) owned by a municipality but in the hands of an operating company.

It would be advisable that municipal corporations coming under the second and third classes be not considered in the report, as they differ radically and essentially from the utilities under discussion. Cost data referring to one class could not fairly be applied to the other.

It has been considered one of the strong points of the Society that each member speaks from his experience, but in this case the speaker regrets that though an intimate knowledge of water-works has been shown, comparatively few of those having a practical knowledge of railroad construction and the detail of maintenance have discussed the subject.

The report abounds in extracts from Court decisions; some of these are good law, others very bad engineering. Where Court decisions are bad engineering, in the opinion of the Committee, it is its duty to set forth the fact clearly in the report. It is safe to assume that the Courts would be greatly influenced by the conclusions arrived at by this Society. The speaker goes further, and does not hesitate to state that this report will influence, not only the United States but the whole engineering and legal world, in so far as engineering applies to law. Therefore, as a Society, we cannot be too careful in what we say in the final report.

These words occur on page 6: "a fair return and no more." What is a fair return? Without discussion, reason, or stated authority, many have selected 8% as a fair return for an investment. It is surprising that this has gone so far without challenge. There is some reference to what are called "losing ventures," which is an admission that, although 8% may be aimed at, a minus quantity may result. Had 8% been the expected profit, the great railroads of the United States would never have been built. Some have stated that losing ventures do not count. The country must have railroad service. The very life of its industries demanded the existing railroads, and the

Mr. Muirhead.

increase of population will demand their extension, whether they are losing ventures or not. Railroads going east and west are said to pay better than those going north and south. Be this as it may, the weak must be carried by the strong. The rates must allow an excess on paying railroads to enable them to form a complete system containing those so-called "losing ventures". The report admits unknowns and probabilities which bring out the fact that "anticipated profits take unto themselves wings". The original risk in building the great railroads was enormous. A few years ago it was assumed by all that the railroads were on a fairly steady financial basis, but things spring up, as it were, in a night, and to-day we do not know where we stand.

The recent action of the Federal authorities in placing the Panama Canal free to the commerce of the coasts was almost a confiscatory action, so far as the railroads are concerned for the next few years. It shows very clearly that the roads had some confidence that this act would be repealed or we would have a worse panic than exists to-day. The fear of adverse or confiscatory legislation must be reckoned with; in truth, it is worse than the prospects of fire, storm, pestilence, or even war. Strikes are becoming a certainty, instead of a possibility. Considering all these risks, the speaker claims that 8% is quite inadequate as a maximum profit to aim at in making our costs for the construction, maintenance, and depreciation of a public utility.

A great many are so accustomed to New York State that their vision does not go beyond it. In such a steady place, 8% is a good investment; but there are first mortgages on agricultural land in a high state of cultivation that can be had in other States at 10 per cent. This being so, can it be expected that the public will be contented or continue to invest in railroads at a possible maximum of 8%, subject to all the risks attendant to-day on unsettled political conditions?

In considering stock, there has been no mention, so far, in the discussion or in the report itself, of the double condition "stocks and bonds". There are a great many utilities in which it is the custom to pay a fixed rate of interest on bonds, or for money actually borrowed by the company, perhaps for an improvement or extension, while at the same time profit has to be made on the utility for the benefit of the owning company on its stock. It is this stock that is so often referred to as "water," a term that agitators have used against the public interests. Sometimes this stock is given for services; at other times it is sold far below its face value. It is often held for years without profit while bonds are being paid off; surely it is worth consideration. The speaker is much impressed by a modern theory that the dollar mark has no place on stock certificates, and is only misleading. He inclines to the theory that the stock certificate should show what fraction of the entire property it represents after bonds, etc.,

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are paid. He firmly believes that, had this system been inaugurated some few years ago, the present valuation would never have been started. It is not out of place to remark that the meaning of the terms, stocks and bonds, is far from being understood by the general public, or even by those individuals fortunate enough to be owners of both.

Possibly the most valuable thing some utilities own is their franchise. The speaker has heard no reference to the value of the franchise. There is some reference to the "physical value" of a utility, but there is not sufficient stress on the fact that the physical is not the actual value. The franchise is often obtained at an enormous cost. Page 22 states: "all necessary permits from public authorities are obtained". To this might be added the words: "often at an enormous cost". It is not just to neglect this in after years. The speaker cannot agree with the remarks on page 20, under the head of unit prices, but inclines to the belief that the unit prices used should be as nearly as possible those expected to prevail during the life of the rates which are adopted as a result of the valuation. Certainly, no price lower than that of material at the date of valuation should be used. Some material, due to the removal of the tariff or to improved methods of manufacture, may be counted on to fall in price, but that is the exception rather than the rule. The prices of some materials fluctuate; then an average would be fair; but the inclination is for prices to keep on increasing. This is particularly true in the case of timber, for the increased cost has been so great that, in spite of an easing up of specification and inspection, its use for many purposes has been discontinued.

On page 22 are the words: "and for many other things". What are these many other things? The things never thought of and never mentioned. One of these things is lobbying. It is necessary to lobby to start a railroad or other public utility, and it is necessary to lobby to keep that utility in life. True, some of these "many other things" are illegal and punishable by fine and imprisonment; at best it is dirty work and must be done, or public utilities would come to an untimely end. Lobbying, being one of the expenses of starting and maintaining a public utility, must in the end be paid for by the public, and it could be placed before the public in no better way than by having it honestly, forcibly, and fearlessly set forth in the limelight by this Society. The standing and the well-known honesty of the Profession make the speaker feel confident that to all it is a matter of disgust, but whether it is or is not, it is there, and in the meantime must be acknowledged under some name or other, and, unless openly acknowledged, the condition will never be improved. Disgusting as this condition is, it must be admitted that worse conditions have been

tackled by engineering bodies and abated with wonderful success. As a rule, engineers are not well versed in these matters, and therefore make insufficient allowances. Mr. Muirhead.

The second paragraph on page 67 is the most surprising part of the report. There are cases where railroad banks have been top-soiled and sodded by contractors engaged for this special purpose. Other banks have been carefully cut and cultured for many years. If, in the Committee's opinion, this has no place in the valuation of the property, the speaker respectfully declares a decided difference of opinion.

Under the head of depreciation the report is academic rather than practical. The methods of depreciation given apply more to the first few years of the existence of a new utility than to the valuation of an old one. For instance, should legislation favorable to the development and application of water-power for traction purposes become law, it would immediately retire locomotives in such numbers that they could not be disposed of. Then again, did not the steam turbine put all its competitors out of business in the generation of electricity? Is not the Diesel engine busy throwing out the steam turbine? What next?

The condition of railroad ties, rails, telegraph poles, etc., should be a matter of exact knowledge, in decided preference to an academic formula.

For many years we have had railroad rates which experience has shown have allowed a fair return on the investment. Recent legislative interference has very much reduced this return, and it is only fair that the rates should be raised. It may be remarked that the whole subject has been illustrated by the Italian railways as reported by the *New York Times*, as follows:

"The session of the Chamber of Deputies of Italy on Sunday gave a good example of the short and easy way to deal with railway wages and rates. Something like 100,000 railway men are threatening a general strike, and the Premier surrenders at discretion. He enlarges the pensions of the employes of the State railways by 11 per cent. He raises the minimum wage, and incurs obligations enlarged by $3,000,000 annually. In the words of the cablegram, 'This sum would be obtained by increasing the railway rates'. It's no harder than that, where Governments own the railways. The Deputies clinched it by a vote of 303 to 122. The reason why it is so easy is that 'the unions of railway men transform themselves at election time into instruments for collecting votes.' Those who like increases of railway rates, and politics dominated by unions, must like the example Italy affords of Government owned and operated railways."

CHARLES KIRBY FOX, Assoc. M. AM. SOC. C. E. (by letter).—The writer has read this valuable report and the discussion on it with a great deal of interest. Though, in general, he is in sympathy with most of the conclusions set forth, especially the so-called Equal-Annual- Mr. Fox.

Mr. Fox.

Payment Method, he is of the opinion that some of these conclusions are deduced from the valuation of minor and somewhat localized utilities. According to his understanding, the Committee was appointed for the specified purpose of formulating principles and methods for the valuation of railroad property and other public utilities. The art or science of valuation, in spite of the abundant literature on some phases of the subject, is comparatively new, and rapidly changing and improving. For these reasons this report should be received by the Society without any endorsement other than to thank the distinguished authors for their careful and painstaking work.

The valuation of utilities for the various purposes is probably the greatest single economic question before the American people to-day, affecting as it does practically untold millions of wealth, the income, wages, and prosperity of a large percentage of our people and to a certain extent the living expenses of practically every inhabitant. Probably the prosperity of no single class is affected as much as that of the civil engineer.

Although this report distinctly refers to valuation for rate-making purposes, the methods and principles involved are practically the same as in the valuation for other purposes.

In the last few years a great many so-called "muck-raking" articles have been published tending to create and mould public opinion to believe that nearly all public utilities are over-capitalized and the stock badly watered. Among well-informed people it is almost universally conceded that American utilities, as a whole, are the lowest capitalized, best managed, most efficient, and cheapest in the world.

It is almost always conceded that valuation is only one of the factors involved in rate-making, and that the rate should earn a fair amount on the capital invested. These two statements do not go together, and the second should be modified in order to take in the element of risk, so that, where a venture is in a new and untried or risky field, and there is a possibility of failure, the rate should be increased to cover this item and give the capitalist a chance to recuperate for other losing ventures in similar fields. That is the most important criticism the writer can offer in regard to this report and practically all other valuation literature.

Mr. Thomson.

T. KENNARD THOMSON, M. AM. SOC. C. E.—The Society is very much indebted to Mr. Stearns and his Committee, for the courage and industry with which they have attacked this stupendous problem.

It seems almost as difficult to try to lay down, in advance, a schedule for a new or even an old corporation, governing the amount that it should set aside each year for depreciation or renewals, so that, at the end of a fixed term of, say, 10, 30, or 50 years, the fund would be sufficient to renew the plant, as it would be to estimate what a young man, just out of college and about to be married, would be worth at

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the end of, say, 40 years, and to order him to set aside a fixed proportion of that amount each year, so that when he was too old to work, or dead, his wife would have enough money in the depreciation account to purchase a new plant or renovate the old one.

The report of the Committee and the various discussions thereon naturally cover a great many phases of the subject, and many pages, to read all of which, by one anxious to obtain information for some particular purpose, would be a very arduous undertaking. It is to be hoped, therefore, that, when the discussion is all in, the entire report will be entirely rewritten, divided into proper headings, and indexed so thoroughly that any engineer, lawyer, or layman can readily obtain the opinions of the Committee or individuals on any branch of the subject.

Mr. Sparrow.

W. W. K. SPARROW, ASSOC. M. AM. SOC. C. E. (by letter).—In a previous discussion* of this report, the writer dealt with the questions of "Going Value" and "Land Values", and now, after further study, proposes to take up the question of "Depreciation and Appreciation."

Probably more has been written on depreciation than on any other question connected with the valuation of public utilities, and the greater part of the discussion on the report of the Committee has centered around this subject. The writer will first deal with the method of computing the amount of the annual depreciation, and then take up the question of whether rates should be based on the depreciated value or the value new.

The Committee has offered for consideration five different methods, and of the five, the writer is in favor of the Straight-Line Method, Value of Plant, Depreciated, because of its greater simplicity both in accounting and in being understood. It is true, as pointed out by the Committee, that the Straight-Line Method calls for a higher annual return in the early years of the property, which is just the time when it can be least afforded, but it is believed that, if regulation of public utilities has come to stay in the United States, public service commissions and other regulating bodies must see that rates are fixed so that *bona fide* new enterprises will be enabled to earn sufficient to set aside a proper and adequate depreciation fund, for otherwise such rates are not fair or reasonable. This being so, the writer believes that this objection to the Straight-Line Method will be eliminated.

The Committee makes the claim in favor of the Equal-Annual-Payment Method that the Straight-Line Method, Value of Plant, Depreciated, requires the rate of return to decrease annually. Theoretically, this is correct in dealing with an imaginary plant which in a definite period depreciates from 100% to 0, but, in actual practice, a successfully operated plant has a perpetual life, and the annual allow-

* *Proceedings*, Am. Soc. C. E., for February, 1914, p. 361.

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ance for depreciation will be used either in replacements, or in additions and betterments, until such time as it is required for replacements. Looking at the matter from this standpoint, it does not appear that of necessity the rates must be changed in order to do justice to the public. For the purpose of illustration, assume that a utility had started with a $100 000 property, and that under regulation the annual return for depreciation had been fixed at 5% per annum. At the end of 10 years, the company would have had paid to it, on account of depreciation, $50 000. Assume that this property had been kept up to a high standard of efficiency and that $30 000 had been expended in replacements, leaving the condition percentage of the property 80. Assume, further, that the remainder of the earnings on account of depreciation, or $20 000, had been re-invested in additions to the property. On the basis of the present value of the property in use and useful service, the company would be entitled to a return on a capital value of $100 000, whereas, if the rates had not been changed on account of the original investment having depreciated 20%, it would be receiving a return on $120 000. This assumption, however, presumes that the investment in additions would yield the same net revenue percentage as the older and fully developed property. This is very improbable. In the case of a street railway, electric light plant, telephone, or water plant, extensions will generally be made to new suburban districts or outlying property which is not built up or thickly populated, and such extensions, on the rates already fixed, will be hardly likely to earn much if any profit.

This illustration presupposes that the earnings on account of depreciation have been used in replacing depreciated property and in additions, and the writer anticipates that under regulation the depreciation fund will be safeguarded so as to insure that it will be used in such a manner and not invested outside the property for the direct gain of the company.

At a recent meeting, before the Public Service Commission of the State of Missouri, of representatives of the various public utilities operating in that State, to consider the adoption of a uniform system of accounts, the great desirability of allowing the utilities to borrow from the depreciation fund, at a reasonable rate of interest, such sums as they might require from year to year for additions, was very strongly impressed on the Commission. It was pointed out that this would save the companies from having to borrow small sums in the open market at a high rate of interest and, consequently, would decrease the cost of the service. Under a strict accounting, there could be no objection to such use of the company's earnings on account of depreciation until such time as required to replace depreciated property of long life, and this appeared to be the view of Mr. McShane, Chief Accountant for the Commission. If the utilities were allowed

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this advantage of obtaining money for additions at a low rate of interest, the public would benefit indirectly, inasmuch as such additions and extensions would be more easily obtained and the cost of the service would be less. Under effective regulation, it is reasonable to suppose that rates will be subject to review at intervals of about 5 years, and, if a uniform system of accounts is in force, it will be very easy for the regulating body to ascertain whether a utility is making more than a fair and reasonable return on the present value of the property, whether due to an excessive return on account of depreciation or for any other cause, and, if so, to adjust matters.

The Equal-Annual-Payment Method, as recommended by the Committee, has the advantage over the Straight-Line Method that it does not require any change in rates, and only allows a return to the utility on the present value of the property. The writer has two objections to this method: first, that, as already stated, it is difficult to explain to non-technical men, which is a great disadvantage, and it requires a more complicated method of accounting; and, second, that the annual payments are not equal unless the interest on depreciation is the same as the rate of return fixed on capital.

Should Rates be Based on the Present or Depreciated Value?—The writer fully endorses the recommendation of the Committee that rates be based on the depreciated value of the property, the Sinking-Fund Method excepted. There has been a great deal of discussion on this question, and the weight of it has been against the recommendation of the Committee, but the writer has not been convinced by any argument in favor of the public continuing to pay a return on a portion of an investment which it has paid back to the company.

On page 32 of the report, the Committee states:

"A depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

This is a very clear, concise, and fair statement, but one for which Mr. Floy, in a recent discussion,* takes the Committee to task.

Mr. Floy says:

"The Committee * * * has developed a unique theory of estimating and providing for depreciation, the allowance therefor being stated to be, 'in effect, a payment from the rate payer to the corporation of a part of its investment'. This is certainly a novel definition, and one that is contrary to accepted practice."

If a company invests $100 000 in a public utility, and if, at the end of 4 years, it has collected from the rate-payer, in addition to its operating expenses and a fair return on its investment, $20 000 for the specific purpose of returning to the company the value of the

Proceedings, Am. Soc. C. E., April, 1914, p. 1129.

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property which has been worn out or partly worn out in its service, what does this constitute? If it does not constitute a return of a part of the company's investment, the writer is forced to admit that his views on accounting are very foggy. To the writer, the novelty is all in Mr. Floy's statement, and if he is correct that in the East such a transaction is not considered a partial return of the investment, then the rate-payer in the East has a good deal to learn from his brother in the West. The United States Commissioner of Internal Revenue, under date of December 15th, 1911, issued a synopsis of decisions relating to the special excise tax on corporations. This synopsis contains the following rule:*

"83. Depreciation to be an allowable deduction in the return of annual net income of a corporation must be charged off on the ledger of the corporation, so as to show a reduction in the capital assets of the corporation to the extent of the depreciation claimed."

If money paid on account of depreciation does not constitute a return of the investment, why does the Commissioner of Internal Revenue specify that capital account must be charged off to the extent of the depreciation claimed?

If Mr. Floy would obtain for publication by the Society the opinion of a competent and disinterested accountant in support of his statement that "money paid by the rate-payer to the corporation on account of depreciation is not in effect a return to the corporation by the rate-payer of a part of its investment", the writer believes that he would perform a service which would be appreciated by many members of the Society.

Mr. Floy labors hard to show that the decisions of the Supreme Court of the United States do not lay down the principle that depreciation must be deducted from the reproduction-value-new in order to arrive at the present value of the property. He construes the decisions in the Minnesota Rate Case to mean that only actual depreciation should be deducted, and not theoretical depreciation. The writer cannot agree with this interpretation of the opinion, but, even if Mr. Floy were correct in his views, it does not seem to the writer to be very material. The real and vital question in any particular case is, what method has been used in computing the annual return for depreciation? If theoretical depreciation, as Mr. Floy terms it, has been included, then theoretical depreciation must be deducted in arriving at the present value. The Committee has very clearly shown that the amount on which a company is entitled to a fair return is inseparable from the method used in computing the annual return for depreciation. This is also emphasized by Mr. Maltbie in his discussion.† If this fundamental principle were clearly understood,

* Taken from Whitten's "Valuation of Public Service Corporations," p. 409.

† *Proceedings*, Am. Soc. C. E., for April, 1914, p. 1126.

a long distance would have been traveled on the road toward a satisfactory solution of this subject. Mr. Sparrow.

Mr. Willoughby, who confines his discussion* to railroad property, is also unable to accept the statement of the Committee that "a depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment." His reasons appear to be based on the fact that the present method of railway accounting charges replacements to maintenance which is an operating expense. Admitting that railways are in a different class from other public utilities, that depreciation on rails, fastenings, cross-ties, ballast, wooden bridges, and trestles, accumulates quickly, and that the annual operating expenses care both for the depreciation and maintenance, does this affect the underlying principle involved in the statement of the Committee? The earnings on depreciation of such short-lived property, where such earnings are put back into the property from year to year, are not involved, and it is purely arbitrary whether such expenditures are charged against depreciation or maintenance. It is plainly impracticable every time a section foreman renews a cross-tie to carry this through the books as a charge against depreciation, just as it would be impracticable in the case of a telephone company to make a similar charge every time a lineman replaced a broken insulator. For convenience such small charges are regarded as maintenance. In the case of property of long life, however, such as, say, the Quebec Bridge or the Pennsylvania Terminal Station, where the company earns each year from the public a depreciation allowance, would it be fair for the company, while the depreciation is maturing for a period of perhaps 50 years, to earn interest on such earnings and, at the same time, collect from the public a return on the full value undepreciated? Apparently, Mr. Willoughby does not think so, as he says that:

"The only injustice which the public can assert is that the rates paid during the period in which the depreciation was maturing have been converted to the use of the owner without an allowance for interest, and possibly have been used for dividends to the owner. It is the opinion of the writer that justice to the public requires an interest allowance during the period the depreciation is maturing * * *"

The writer agrees with Mr. Willoughby that if this were done the public would suffer no injustice, provided the rate of interest on such depreciation were the same as on capital. To the writer, however, the same principle is involved in Mr. Willoughby's statement, unless he misunderstands him, as in the statement of the Committee, for the only reason for the company to set up in its accounting such a liability recognizes the fact that it is only holding in trust such earnings

* *Proceedings*, Am. Soc. C. E., for April, 1914, p. 1117.

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from the public during the period the depreciation is maturing, and that, when matured, these earnings will be used to restore the original investment.

Mr. Willoughby will no doubt agree that the normal condition percentage of the physical property, excluding land, of a well-managed railroad which has been in operation for some 20 years, is about 80%, and that it would be impracticable, of not impossible, to maintain it at 100 per cent. This being so, what has become of the remaining 20 per cent? Either the railroad company has collected it from the public over and above its operating expenses and a fair return, and has paid it back to the stockholders in dividends or reinvested it in other property, or it has not. In the former case, it has most certainly had 20% of its original investment returned to it, and it is clearly not entitled to earn a return on it still, and, in the latter case, it would certainly be an injustice and confiscation of property to deny the company a return on it. The fair thing to do would be to allow the company to amortize the amount in a given period.

A number of writers who argue in favor of the rate of return being based on the value of the plant undepreciated, build up their case on the foundation that the service the public is receiving from the corporation is just as good or, in fact, better in the case of an old and seasoned plant. Although this argument is plausible, it is fallacious and inconsistent, for the reason that service is not the basis on which we started to determine the fair rate of return. The Courts have ruled, and it is, at present, an accepted fact, that the fair rate of return shall be based on the present value of the property in the use and useful service of the public. This being so, and having started on this theory, surely it is inconsistent and improper to switch to another theory based on the "value of service." If value or worth of the service is to be the criterion on which a return is to be fixed, why go to all the expense and trouble of making a physical valuation of the property? If a street-car company, in, say, Milwaukee, is charging 5-cent fares with universal transfers, and this has been established as a fair and reasonable charge for this class of service, it would be entirely unnecessary, in, say, the case of Kansas City, to make a physical valuation. If service is the basis, then the only question is, is the service in Kansas City up to the required standard? and it would be immaterial whether in the case of Milwaukee this charge yielded a net return of 7% and in Kansas City 12 per cent. Such a method of fixing rates would be very unfair to the rate-payer, and would involve a very dangerous principle which would open up an avenue to fraud and imposition at the expense of the public. To illustrate this, suppose a corporation invested $10 000 000 in a utility and at the end of, say, 5 years, had collected from the rate-payer, on account of depreciation, $2 500 000 (being at the rate of 5% per annum) and of

this sum had only expended in replacing the depreciated property $500 000, then the depreciated value of the property would be 80% or $8 000 000. At this stage the original company decides to sell the property and, on the perfectly legitimate basis (assuming that the rates are based on service and not on present value) that the investment is yielding a fair return on $10 000 000, sells it for this amount. The net result is that the original company retires with its original investment of $10 000 000 and $2 000 000 to boot, which has been collected at the expense of the rate-payer to replace depreciated property. Mr. Sparrow.

No doubt this will be considered by many to be a far-fetched supposition, but in view of some recent manipulations, disclosed by investigations of the Interstate Commerce Commission, where railroad manipulators bought properties which they unloaded on their unsuspecting stockholders at huge profits, it is not perhaps so improbable. At all events, it would tend to show the unfairness and absurdity of fixing the rate of return on anything but the present depreciated value.

Adaptation and Solidification of Roadbed.—On this question the Committee, in its report (page 66), makes the following statement:

"A railroad roadbed, built of earth that has been dumped without consolidation will, as the result of running trains and the effect of weather conditions, settle, and earth will be washed from the fresh slopes, so that additional earth, ballast, and labor will be required to produce a solid and full-sized embankment. All such costs required to complete machines and structures are properly chargeable to capital account, and if the unit prices used in making the valuation do not cover these items, they should be added."

The writer cannot accept this statement, which is, pure and simple, a recommendation that operating expenses be capitalized. It might possibly be argued that in the early life of the property the earnings did not yield a sufficient return to pay such operating expenses and at the same time pay a fair return. This, however, is cared for in the Committee's recommendation that such losses shall be capitalized as development expenses. If the company has earned a fair return and paid for such adaptation and solidification of the roadbed out of operating expenses, it has no right to expect to capitalize such expenses. If it has not been able to earn a fair return and pay these operating expenses, it will, under the Committee's recommendation, be allowed to capitalize them under development expenses; and, of course, it would be improper to capitalize them twice.

J. P. Snow, M. Am. Soc. C. E. (by letter).—In the discussion of depreciation methods published in the *Proceedings* of May, 1914, the writer says, at the bottom of page 1390: "The final outcome at the end of the life cycle is the same if interest is fully allowed for in both cases." That is, the straight-line and equal-annual-payment Mr. Snow.

Mr. Snow.

TABLE 33.—COMPARISON OF FOUR METHODS OF COMPUTING DEPRECIATION.

Years.	No. 1. Straight-Line Method.				No. 2. Sinking-Fund Method.				No. 3. Equal-Annual-Payment Method.				No. 4. Replacement Method.			
	Capital on which return is computed.	Return on capital.	Depreciation allowance.	Combined return.	Capital on which return is computed.	Return on capital.	Depreciation allowance.	Combined return.	Capital on which return is computed.	Return on capital.	Depreciation allowance.	Combined return.	Capital on which return is computed.	Return on capital.	Depreciation allowance.	Combined return.
1	100.00	7.00	10.00	17.00	100.00	7.00	7.24	14.24	100.00	7.00	7.24	14.24	100.00	7.00	0.00	7.00
2	90.00	6.30	10.00	16.30	92.76	6.49	7.75	14.24	92.76	6.49	7.75	14.24	100.00	7.00	0.00	7.00
3	80.00	5.60	10.00	15.60	85.02	5.95	8.29	14.24	85.02	5.95	8.29	14.24	100.00	7.00	0.00	7.00
4	70.00	4.90	10.00	14.90	76.73	4.75	8.87	14.24	76.73	4.75	8.87	14.24	100.00	7.00	0.00	7.00
5	60.00	4.20	10.00	14.20	67.86	4.09	9.49	14.24	67.86	4.09	9.49	14.24	100.00	7.00	0.00	7.00
6	50.00	3.50	10.00	13.50	58.38	3.38	10.15	14.24	58.38	3.38	10.15	14.24	100.00	7.00	0.00	7.00
7	40.00	2.80	10.00	12.80	48.23	2.62	10.86	14.24	48.23	2.62	10.86	14.24	100.00	7.00	0.00	7.00
8	30.00	2.10	10.00	12.10	37.36	1.80	11.63	14.24	37.36	1.80	11.63	14.24	100.00	7.00	0.00	7.00
9	20.00	1.40	10.00	11.40	25.74	0.98	12.44	14.24	25.74	0.98	12.44	14.24	100.00	7.00	0.00	7.00
10	10.00	0.70	10.00	10.70	13.31		13.31	14.24	13.31		13.31	14.24	100.00	7.00	100.00	107.00
Totals	100.00	138.50	100.00			72.38	142.38			142.40	100.02			170.00	100.00	

Mr. Snow.

methods. To show that the result is identical for the four methods: straight-line, sinking-fund, equal-annual-payment, and replacement, when full interest is allowed, Table 33 is given, indicating the life history of a single 10-year item, the value of which, new, less scrap value, is represented by 100. In all calculations 7% is used. Mr. Snow.

The depreciation allowances in methods Nos. 1 and 3 are in the nature of partial payments on a note, totaling 100 at the end of the term; and hence annual return on capital must be cast on the remaining value, as is customary with the ordinary promissory note. In method No. 2 the allowances do not sum up to 100, but must be allowed to accumulate at compound interest when, at the end of the term, they will amount to 100. This is comparable to the common method of paying bonds by a sinking fund, and, of course, the annual capital return must be paid on full value new.

In method No. 4 no replacement fund is provided, but the money for replacement is raised when and as needed.

These four methods are just and equitable to all parties concerned, if properly applied, and the amount paid by the public on account of capital as combined return is exactly the same by each method, if the computations are correctly made and full interest is allowed.

The amount given in the columns of combined returns are due at the end of the year at which they are given or at the beginning of the following year. If these amounts are figured at compound interest, at the rate assumed, to the end of the term, that is 10 years for the first term and so on, it will be found that each of the four columns amounts to 196.68 if sufficient decimals are used.

This demonstration is plain arithmetic. It is a theoretical case, to be sure, but it typifies any one item of a plant, and a whole is but the sum of its parts. There is surely no more confiscation of property by one method than by another, as has been harshly claimed by some critics of the Committee.

As Whitten says: "Depreciation is a question of cost accounting." Each method requires its own system of bookkeeping, and such features as equality of rates from year to year, the feasibility of using depreciation funds regularly for replacements, and the protection of the depreciation funds from distribution as dividends, would seem to be the governing factors in choosing the method to be used.

E. W. James, Assoc. M. Am. Soc. C. E. (by letter).—A perusal of the discussion invited by the Report of the Committee emphasizes the point raised in the writer's previous comment.* There has been neglect of some fundamental matters that should be disposed of before details of methods, accounting and other forms, and treatment of peculiar or special problems are finally settled. Mr. James.

It is too much to say now that the discussion is chaotic, although

* *Proceedings*, Am. Soc. C. E., for February, 1914, p. 351.

Mr. James.

it has long had that quality in the engineering press, for the work of the Committee certainly has ample merit of form and inclusiveness to fix the lines of discussion much more definitely than they have ever yet been fixed. The discussion, however, serves to indicate that some matters of importance have been passed by, and until these matters are duly considered and the Committee issues some decision regarding them, there will remain work to do in establishing the report on the high basis of principle that should result in its general acceptance as the dicta of the Society.

For instance, a point has been raised regarding the very creation of the Committee. Now, the appointment of the Committee was under the following title: "A Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities."* The subjective form of this title does not include any reference to rate-making, but the circumstances attending the work of the Committee, and the fact that the valuation of public utilities, more particularly of railroads, has become important because of the avowed purpose of using it in rate-making, certainly makes rate-making cognate to the work of the Committee. This does not appear to be the opinion held by all, however, for both Mr. Phillips and Mr. McCarty would circumscribe the work of the Committee closely. Mr. Phillips questions the advisability and the precedents of using valuations for determining or adjusting rates, and Mr. McCarty would confine the study to the strictly technical details of a physical appraisal, and omit all consideration of the legal, economic, or sociologic details of the problem. The writer would not presume to decide, as between the Committee and these gentlemen, but the circumstance indicates that the very purpose, the *raison d'être*, of the Committee's action is still at this stage a matter about which there are differing opinions. Again, Mr. Lavis suggests that it was unfortunate that any reference was made to railroads in the title of the Committee. It would seem that 19 months is a sufficient time to establish such matters as these, if not to the satisfaction, at least to the understanding, of all.

The writer has already called attention to the existence of important underlying principles connected with the problems of valuation for rate-making, or for any other purpose, and Mr. Dana and others recognize this same fact, but although the Committee has made a substantial addition to the general consideration of the subject, it has not taken advantage of an excellent opportunity to formulate these principles or to define terms.

The original report, however, is doubtless only a consideration in part of the general subject assigned to the Committee for investigation, and, if so, there will still remain an excellent opportunity to consider

* *Proceedings*, Am. Soc. C. E., October, 1911, p. 479.

Mr. James.

other aspects of the general question. It is to be hoped that the Committee will be continued until it has completed such studies as will serve to elucidate the fundamental principles of valuation as applicable to rate-making, to taxing, and to acquiring by purchase. The general title of the Committee permits this to be done. There is no doubt that such a task—for task it certainly would be—will be attended by valuable results.

Without going to any great length in supporting the question, the writer wishes to point out certain actual conditions obtaining to-day in the valuation discussion.

There is a lack of definition. Although this phase of some branches of engineering has been rather overdone recently, no mistake can be made in stating clearly what is meant by cost-of-reproduction-new, original-cost-to-date, depreciation, deterioration, depreciation-reserves, and the commoner terms used in connection with valuation matters.

For instance, the exact meaning of cost-new (or cost-of-reproduction-new) should be considered and stated. The Committee might state it thus: The term "cost-new" is to be used instead of "cost-of-reproduction-new," and it will be considered as the cost to reproduce the plant in all its physical elements or their respective equivalents as of the date of valuation, assuming that the plant is non-existent on its present site, and that the construction was begun at a sufficiently early date to reach completion at the date of valuation; and all unit prices of labor and materials, and all attending conditions, shall be assumed as of that period only.

Though the Committee would probably not adopt such a wording as the writer's, it could define such a valuation in its own terms, and thenceforth valuations on the basis of cost-new, if made as prescribed by the Society, would mean only one thing. At present, a cost-new valuation may or may not include intangible values; unit prices may be assumed on several varying bases; practically a substitute plant on a different site, bearing little or no resemblance to the existing plant, either in design or in its physical elements, may be evaluated, which, however, would produce an equivalent service; piecemeal, or continuous, construction may be assumed, in an attempt to approximate conditions under which the existing plant was actually constructed, thus confusing original cost and cost-new. If the Society, through its Committee, defined cost-new, the method could then be referred to and used as succinctly as are specifications now prescribed by the Society.

Depreciation and deterioration should be carefully defined and distinguished. Depreciation should be defined with reference to the time element involved, as an accounting term merely. Depreciation begins at once and progresses according to time elapsed, the rate for any period being approximately determinable. Deterioration, however,

Mr. James.

has reference to efficiency of service, and has no necessary connection with depreciation. As in the case of an automobile, the efficiency of the machine may be considerably higher at the end of from 1 to 6 weeks than when brand-new, and consequently the deterioration for that period is nil. Deterioration may then practically halt for several months, or even years. Deterioration has no determinable time element. It affects depreciation as an accounting element only by accident; then the unexpected happens and rapid deterioration occurs, practically compelling the replacement of a part before the estimated time for complete depreciation has elapsed.

Similarly, original cost, depreciation reserves, and other details of valuations should be defined. These definitions cannot be made satisfactory to all, but, if it is borne in mind that adjudication by the Courts will in nearly all cases be had before the fair value for rate-making purposes is fixed, and if the engineer remembers that his figures are not necessarily to include adjustments to establish equity and fairness as between the public and the owners, then the value of determining definitions for the fundamental exhibits submitted to the Courts for judicial review, will be more apparent. The engineer can appear before the Court or commission in support of various adjustments; that, for instance, certain elements of the cost-new valuation are peculiar in a given case and do not represent a fair value from the point of view of the owners; that, in certain cases, the Court should give greater weight to cost-new than to original cost; and, that certain intangible values should be specially considered. Original cost should include actual cost of land, should not include bonds guaranteed by counties or States, nor lands granted by the Government; cost-new, of course, would show land at present-day prices, with accumulated increment; values representing the gifts of land or funds would also show. The Courts would presumably determine on a fair value, using such elements of each valuation, together with other considerations, as might seem just and reasonable.

The writer has elsewhere stated at some length his ideas as to the economic and ethical considerations involved, which Courts should recognize in establishing fair values for rate-making purposes. In regard to all these details, the engineer should be informed; but it is hardly within his province to act in a judicial capacity in making physical valuations. His part is to state, item by item, the physical elements of a property, and assign to each a value. These should be classified and grouped in such a way as to show separately those items that are commonly involved in Court decisions; so that, for instance, physical value, intangible values, unearned increment, contributed land, guaranteed bonds, etc., can be quickly separated. The further part of the engineer will be to understand the relative importance of these items as they might affect a fair valuation for

rate-making in the case in question, and be prepared to defend the inclusion or exclusion of certain of them. Mr. James.

The most successful work of the Committee, it seems to the writer, will be to provide a set of principles to guide the engineer in making valuations according to the various methods, now more or less confused and conflicting.

That no established rules are being followed by the engineers is quite obvious to any one studying the subject. The commonest source of differences lies in the effort of the engineer to do justice in the premises. He cannot do it. It is doubtful if the Courts can. Under the circumstances, a rational course presents itself only when the engineer is sought to fix the value of the several items entering into a property and to give the commission or Court of final adjudication the benefit of his expert knowledge. In the end the Court will determine the fair value for rate-making purposes, taking into consideration, no doubt, numerous economic, legal, sociologic, and other special elements. The exact relation of each of these elements toward one another and toward the general whole will doubtless in time be determined by a series of Court decisions. The engineer is concerned in securing equitable decisions, on grounds as fair as possible to owner and user, in no professional way, and only as all good citizens are concerned. If called on to act as referee, as commissioner, or as final adjudicator, in any case, then he acts in an extra-professional capacity as a judicial officer. The wisdom required for his decisions in such cases will far transcend that demanded by the daily practice of his profession, and a knowledge of many things quite apart from engineering principles will be required.

George L. Dillman, M. Am. Soc. C. E. (by letter).—The "value" of a public utility, like the value of everything else, is largely a matter of opinion, and will be as various as the valuators. Neither is it constant with each valuator for any length of time. Next month, or last month, his opinion will be, or was, different from his opinion to-day. Mr. Dillman.

A valuator whose opinion is constant is necessarily wrong most of the time. The attempt to be consistent makes us liars to the extent that we attempt it. Every man has a point of view different from every other man, which is also changing more or less rapidly.

This is so by reason of the continual change in all the elements of value. Cost of material, cost and efficiency of labor, efficiency of superintendence, availability of specialists, as well as the valuators' knowledge or ignorance of these things, all affect value, and all are constantly on the move.

Actual investment is an element more or less known, and has some bearing, diminishing with time and increasing with knowledge of conditions at the time the investments were made; but actual investment

Mr. Dillman.

is the measure of value of nothing else. Why should it specially apply to public utilities? Why should an investor in a public utility be guaranteed against loss or deprived of unearned increment, any more than the purchaser of a farm?

Value of securities is an element to be considered, but is no proper measure of value. The market for securities is so easily manipulated that there is hardly anything more variable than stocks and bonds. Even without manipulation, the value of securities is only an expression of opinion of that part of the public making the investments or contemplating it. Public opinion, though seldom well expressed, is always wrong. It is not as far wrong as is possible to individual opinion, but it is never right.

Cost-of-Reproduction is an element to be considered. It is logically the best measure of value. Cost-of-Reproduction in present condition of parts being impossible, it becomes necessary to resort to Cost-of-Reproduction-New-Less-Depreciation, as the best method of approximating value. So this favorite method of Courts and commissions is founded on reason. After all, the final test of trust is reason.

A public utility is best operated as a monopoly, and generally is. So the value of the privilege, the franchise, may have a definite value. It may have cost something. It may be a contract that is worth something, though that something may be less definite than the value of physical property. To the extent that it has value, such should be added. This value is more or less tangible. When a franchise has cost nothing, is worth nothing, does not exist, or has competition, it is hardly an element of value.

Preliminary costs, water rights, power rights, promotion, administration, engineering, interest and taxes during construction, are all sufficiently tangible to be added to the above. They should certainly be added as legitimate elements of value. They can no more be fairly excluded than a pumping plant or a terminal or a sub-station.

Anything, in fact, that would cost something in replacement, should be considered and valuated, just as the physical parts are valuated at what they would cost at the time of valuation.

The sum of all the above is the value of the utility, when the utility performs its purpose and is not overbuilt. Every reader has some knowledge of cases where a railroad is constructed with insufficient traffic, a water-works with an insufficient supply, and all utilities with a dearth of customers. In these exceptional cases, the works are worth less than this sum. In no case, however, is the value greater than this sum.

This question of valuation is important to-day, and is growing. If the Society can reach a rational conclusion, the foundation of a great variety of correlative questions will have been settled. It is more important that the basis of valuation be fixed than that it be right.

Mr. Dillman.

If it can be fixed right, well and good, but it will be pretty near right now, and we need it fixed that other problems based more or less on valuation can be considered. So far as can be judged on the Pacific Coast, the Legal Profession is willing to adopt our conclusions if we conclude.

Valuing public utilities has developed a large and somewhat plausible school of "experts," whose efforts are to increase the above valuation by large amounts. Sometimes they have "delivered the goods." Demand, as usual, creates a supply. If they can "deliver the goods," there is a market for them, with fees in accordance.

The main argument is for a large amount for "going value" or "going concern" or "going concern value," which is variously defined, defended, and contended for.

If the valuations are made as above outlined, these intangibles are all covered. The valuation is made as a "going concern." Otherwise, the plant would have a second-hand and scrap value. So the value of "going concern" is entirely included, and is a very large part of the "value," as found above. If anything more is added for "going concern," it is a duplication.

The writer would make a distinction between depreciation and allowance therefor. Two points in depreciation are known: the start (100% value), and the finish (scrap value). As these, like other elements of value, are more or less (principally more) matters of opinion, it can be a matter of small moment, either to the utility or the public, what line is followed in making allowances for depreciation, provided such allowances are considered in rate-making.

No method for such computation will be in accord with the facts, so the Straight-Line Method is advocated, because of its simplicity. If any curve is used, the writer suggests the conic parabolic as being more likely to accord with the facts than the Sinking-Fund Method, and simpler to apply.

The writer would take issue with the Committee's recommendation that present prices and original conditions be used, and prefers present prices, and present conditions. These are better known, and the original conditions are generally a matter of hearsay. Similarly, whether the plant be constructed piecemeal or otherwise is irrelevant. Such may have resulted in economy instead of expense. Certainly, the method used makes no difference in service, or present cost, or depreciation, and, therefore, none in value. The time and the methods assumed in the appraisal should be entirely reasonable for the reproduction now.

The writer can see no reason for finding a different value for different purposes, provided the public service is the same. If other property is assessed for taxation at 60% of its value, 60% should be used for taxation of the utility, but the value should not be changed except on the assessor's books. So, too, portions not used or useful

Mr. Dillman.

might be excluded for rate-making purposes and be included in purchase or condemnation, but the same item in use has the same value for all purposes, and the value of the utility is the sum of the values of its parts.

Mr. Wheeler.

WALTER S. WHEELER, M. AM. SOC. C. E. (by letter).—The writer agrees with the Committee that "the inventory should be made in much detail and with much care in order to obtain the best results". Under the heading "Inventory" the Committee says:

"Take, for an extreme instance, the dredged channel of the Panama Canal leading from the deep water on the Atlantic side to a short distance beyond the shore line. The Annual Report of the Isthmian Canal Commission to June 30th, 1912, states that the geometrical quantity represented by the excavation of this channel to that date was 10 169 000 cu. yd., while the actual quantity taken out of the channel amounted to 22 886 000 cu. yd., an excess of 125 per cent."

Now the writer does not wish to criticize that statement, as he has personal knowledge that it is practically correct, even if he had never seen the Report of the Isthmian Canal Commission. However, he does wish to say that the statement as it stands is very misleading, and the Committee should not have used it without going into the reports more fully.

That report states also, that the unit cost of dredging those 22 886 000 cu. yd., taken as a whole, was $0.2412 per cu. yd., or $5 520 103.20 for the total cost.

The reports also show that the sea-going suction dredge, *Caribbean*, took out the most of that excess of 125%, which was silting (and could have taken it all out), at a cost of approximately 10 cents per cu. yd., that is, 12 717 000 cu. yd. at 10 cents equals $1 271 700.00 for the total excess. Subtracting this last amount from the total cost, we have, $4 248 403.20, which is the actual cost of dredging the original or geometrical quantity of 10 169 000 cu. yd. (within the limits as stated), a considerable portion of which was rock that had to be blasted and dredged at depths varying from 20 to 42 ft. below sea level, with dredges that could not operate in the roughest weather as could the *Caribbean*. This gives an excess of material of 125%, but only 30% in excess of cost.

Thus, on further examination, it is shown, in the case mentioned, that it is not such an extreme instance at all, and that it would have been much fairer, when taking quantity without cost into consideration, to have used the geometrical quantity represented by the excavation for the comparison, than to take the whole quantity excavated. All of which goes to show that the examination of this most important subject cannot be too thorough.

The importance that is given to the Court decisions by those discussing the Committee's report is interesting.

In closing his discussion,* Mr. Knowles says: Mr. Wheeler.

"Reservoirs, pipe sizes, * * *, all need to be planned in varying degrees for the probable needs of a number of years. This is but ordinary good business judgment, and the sooner we can get the Courts to recognize this principle the better for all."

On the same page, Mr. Leighton limits his discussion to citing a part of the discussion of Mr. Hazen and directly after that he cites an interesting Court decision which tends to disprove Mr. Hazen's part of the discussion, just as if the fact that the Court having said it was sufficient proof to be final without further discussion.

The writer is of the opinion that Mr. Stearns "strikes the nail on the head", when he says in his discussion:†

"There is no doubt that a Court is fallible, and that, in many matters, it is dependent on the testimony and arguments brought before it, but when an able Court, like the Supreme Court of the United States, composed of men who are able to take a broad view of matters which are presented to them, makes a decision to which it adheres consistently for many years, such decision should be deferred to * * *."

Mr. J. N. Dodd, in his discussion‡ states that if "80% of the estimated life of the main has expired at the date when the street was paved, only 20% of the added value due to the paving can be added to the capitalization of the company", which infers that if the main was put in directly before the paving was laid, the full value due to the paving should be added.

In most well-regulated cities the franchise ordinances require the companies to lay what mains they will need, or will probably need, for a number of years, in streets about to be paved, and in other cities where such conditions are not included in the franchise ordinances, the companies with keen business foresight will endeavor to do so. According to Mr. Dodd's reasoning, such companies should have the full increased value, due to the paving, added to the capitalization at once.

Those newly laid mains would cost practically nothing to maintain for many years to come, while the old mains of another company, the capitalization of which had been added to by only a proportion of the value due to the paving, would, at times, be leaking gas or water into the street, causing extra expense to the company for repairs, and annoyance to the public.

The estimated life of the mains for proportioning this increased value would be a very variable quantity, especially in cities where electrolytic action is at all active, and in such cases, the added value to the capitalization would, invariably, be too large.

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* *Proceedings*, Am. Soc. C. E., May, 1914, p. 1386.

† *Loc. cit.*, p. 1402.

‡ *Loc. cit.*, p. 1380.

Mr. Wheeler.

Appeal to the Courts for damage from electrolysis is not always successful, due to the owner of the pipes not being able to prove whose electricity damaged them, especially where several electric companies are distributing in the immediate vicinity.

The owner of extra large mains, wisely put in before the new pavement was laid, should not have his property excluded because it is unused, or partly unused, as it is not only a benefit to the public not to have the new pavement torn up, but it may have been mandatory in his franchise ordinance. Also, his mains will deteriorate in value before they are used to their full capacity, and it certainly would be unjust not to allow him increased value due to the paving if his property was excluded on account of unusual size at the time of placing it.

However, in such a case, the writer is of the opinion that if the owner is allowed full value for the partly unused portion of his property, he should not be allowed increased value due to paving. Also, if the owner of the old mains below the new pavement had used ordinary business foresight, he would have renewed his mains before the pavement was put in, and thus saved himself most of the extra expense of tearing up and replacing the pavement for repairing leaks. In the latter case, the annoyance and extra expense to the public should be considered, and he should not be allowed increased value due to paving.

In many of the franchise ordinances granted for street railways in different cities, we find conditions that read somewhat as follows:

"The grantee, its successors or assigns, shall, from time to time, grade or regrade, pave or repave, * * * the right of way granted by this franchise, at the same time that the City shall grade or regrade, pave or repave * * * the adjoining portions of the street. The pavement between the rails and the tracks and for 2 ft. on either side thereof (the grantee's portion), shall be of the same general material as that of the adjacent portion improved by the City, and shall be maintained in an equally good condition as the City's portion."

In the franchise ordinances of the owners of mains and conduits, we find that the owner must raise or lower his property to conform to the new grades of the streets, but we seldom find a condition that requires him to pave and repave and maintain the pavement above his mains and conduits, except temporary openings for repairs, or when the pavement has been put down before the mains or conduits were laid.

If the owners of mains and conduits wish the full value due to the paving added to their capitalization, why not place the condition in future franchise ordinances, requiring them to pay the City for the paving of that portion directly above their mains and conduits, in proportion to the width of the latter, and for a few inches on each side thereof? In such a case there would be no question about adding the value of

the pavement to their capitalization. In most cases it would be impracticable to allow them to do their own paving, as such width would usually be small as compared to the width of the street railway paving, but it seems to the writer that if they desire it, they can be allowed to pay the City for such portion, and it would be just as reasonable as requiring the street railway to pave its portion. Mr. Wheeler.

After reading a number of the discussions on the Committee's report as regards railroads, the writer would suggest that we get away from the atmosphere of New York City and vicinity for awhile. Not but that the railroads having terminals in New York City are vastly important, but no one, so far as the writer has observed, has discussed railroads under this report with examples from any other standpoint. If there is true competition between such railroads, as most of the discussions infer, is not that in itself a regulator of rates until such competition ceases to exist?

The writer is of the opinion that valuation should be studied more thoroughly in the case of monopolies where there is no competition. Mr. Byers gives a very able discussion* from the standpoint of competitive railroads, but he overlooks the fact that some railroads are not competitive and that others serve large territories where there is practically no competition.

The examples which he gives of Eastern railroads, together with his statement that "probably without exception, every railroad serves some territories that are not served by any other railroad", gives one the impression that he considers all railroad business competitive with small exception. Take, for example, our transcontinental lines, and leaving out of consideration the through business from the Pacific Coast to Mississippi River points, do not these lines serve large territories without competition? Also, the railroads of Alaska which are not competitive, but whose original costs were enormous.

In referring to the equation:

$$\text{Tariff Rate} = \frac{\text{Fair Value} \times \text{Fair Rate of Return} + \text{Cost}}{\text{Output}},$$

Mr. Byers states that:

"It is clear that, in applying this equation to a gas plant or other utility of that sort selling one commodity, it is comparatively easy to solve, even though the various factors seem to be not of the most definite. When it comes to applying it to a railroad, the speaker must confess that he would like the Committee to go into a little detail as to the practical application of its theories, * * *."

As we go farther West to locations where competition in the railroad business is not so keen, the unknown quantities represented by

* *Proceedings*, Am. Soc. C. E., May, 1914, p. 1450.

Mr. Wheeler.

the thousands of outputs and tariff rates would be greatly reduced and the equation, therefore, would be easier to apply.

Mr. Byers evidently assumes that the equation could be used on the lines having the greatest competition, for he states:

"There is nothing said, in the Committee's list of principles, about the necessity for first valuing and establishing rates on all competing lines before attempting to value and establish rates on the line which is the immediate subject of the inquiry."

The valuing and establishing of rates on competing lines would certainly have to be considered in connection with the establishing of rates on any particular line in order to arrive at fair conclusions.

Another item that would warrant great consideration is where the bulk of the output is in one direction. A railroad located under these conditions would be justified in charging a higher rate than one located in territories where the cars would generally be loaded in both directions. For example, in years when the price of wheat was high, and the farmers along the Canadian Pacific Railroad were growing practically nothing but wheat, it was raised in such large quantities that the railroad could not take it all away, and the farmers burned much of it for fuel during the following winters. The cars carried wheat in one direction and returned empty.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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HUACAL DAM, SONORA, MEXICO.

Discussion.*

BY MESSRS. CHARLES W. SHERMAN, M. M. O'SHAUGHNESSY, L. R. JORGENSEN, AND L. J. MENSCH.

Mr. Sherman.

CHARLES W. SHERMAN, M. AM. SOC. C. E. (by letter).—The writer has long felt that the arched dam is worthy of much greater attention than it has usually received. No one has ever shown how such a dam could fail, if the abutments were sound and the crushing strength of the arch and abutments was not exceeded; and, as a matter of fact, as noted by Mr. Hawgood, there does not appear to be any record of failure of a dam constructed on this principle.

The total number of dams in which dependence is placed on arch action for stability is so small that it seems desirable that a complete list be prepared. The author has overlooked several American arched dams, two of which rank next to the Bear Valley and Upper Otay in boldness, as measured by intensity of stress in the arch. The dams referred to are those at Lewiston, Idaho, and Winchester, Ky., built by William Wheeler, M. Am. Soc. C. E.

Table 9 contains a complete list of arched dams, as far as the writer has been able to ascertain, and includes data on ten dams not given in Table 1.

It is particularly interesting to note the test of a brick arch, 4 in. thick, with a radius of 58 ft. 1 in., comprising a facing wall of the Ithaca Dam, which occurred during the construction of the dam, as described by Gardner S. Williams, M. Am. Soc. C. E.† The arch was 52 in. high, with a clear span of 90 ft. and with a head of water

* This discussion (of the paper by H. Hawgood, M. Am. Soc. C. E., published in April, 1914, *Proceedings*, and presented at the meeting of May 20th, 1914), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† *Transactions*, Am. Soc. C. E., Vol. LIII, pp. 199–200.

TABLE 9.—PRINCIPAL DETAILS OF CURVED MASONRY DAMS.

Reference.	Place.	Maximum height, in feet.	Thickness at base, in feet.	Thickness at top, in feet.	Maximum stress in arch, pounds per square inch.	Radius of up-stream face, in feet.	Length at top, in feet.	Character of rock.	Date.
(1)	Katoomba, N. S. W.	25	20.29	3.0	233	220	320	Sandstone	1905
(6)	Cascadilla Cr., Ithaca, N. Y.	25	40.0	2.5	189	70	96	Shale	1904
(1)	Picton, N. S. W.	28	13.62	7.0	186	120	112	Sandstone	1897
(3)	Winchester, Ky.	31	8.58*	4.83	498	318.4	407	....	....
(1)	Qn. Charlotte Vale, N. S. W.	32	8.65	3.0	155	90	113	Quartzite	1898
(1)	Parkes, N. S. W.	33.5	13.5	3.0	373	300	540	Granite	1897
(1)	Lithgow No. 1, N. S. W.	35	10.88	3.5	155	100	178	Sandstone	1896
	Barren Jack, N. S. W.	38	5.00	....	264	80	....	....	1907
(12)	Meer Allum, India	39.0	8.5	....	164	82	178	....	1800±
(1)	Wollongong, N. S. W.	42	11.62	3.5	311	200	535	Basalt	....
(1)	Cootamundra, N. S. W.	46	13.0	3.0	389	250	640	Granite	1898
(1)	Wellington, N. S. W.	48	10.0	3.0	311	150	350	Conglomerate	1899
(2)	Bear Valley, Cal.	48 (total 64)	8.4 at 48 ft.	3.17	825	335	300	Granite	....
(1)	Mudgee, N. S. W.	50	18.0	3.0	311	253	498	Altered slate	1899
(11)	Las Vegas, N. Mex. (built)	50	15.50	4.0	350	250	210	....	1910
	" " " (proposed)	95	43.30	4.0	300	250	390	Sandstone	....
(1)	Parramatta, N. S. W.	52	15.0	4.8	223	160	225	Sandstone	....
(3)	Lewiston, Idaho	55.5	14.5	5.33	475	286.5	288	....	....
(8)	Crowley Cr., Malheur Co., Ore.	60	5.2	3.0	350	70	170	....	1908
(1)	Tamworth, N. S. W.	61	21.5	3.0	311	250	440	Granite	1898
(9)	Goodwin Dam, Stanislaus River, Cal.	61 (total 70)	12.0	8.0	297	135	233	....	1913
(1)	Medlow, N. S. W.	65	8.96	3.5	186	60	124	Sandstone	1906
(7)	Upper Otay, Cal.	75	14	4	604	359	350	....	1900
(1)	Lithgow No. 2, N. S. W.	87	24.0	3.0	155	100	221	Sandstone	1906
(10)	Huacal, Sonora, Mex.	88.5	12.83	3.5(?)	226	76	140	Andesite	1912
(2)	Sweetwater, Cal.	90	46	12	188	222	380	....	....
(4)	Six-Mile Creek, Ithaca, N. Y.	90†	7.75	....	288	67.85	....	Shale	1903
(2)	Barossa, S. Australia	95	84	4.5	242	200	....	....	1903
(2)	Zola, France	119.7	41.82	19.02	196	158	205	....	....
(4)	Lake Cheesman Dam, Colo.	210 to H. W.	176 72.66 at El. 100	18	....	399.89 at top	....	Granite	1900
(5)	Pathfinder, Wyo.	210	94	10	181	150 at center 186.5 at base	....	....	....

(1) *Engineering News*, May 19th, 1910.
(2) Wegmann's "Dams," 5th Edition.
(3) Data obtained from William Wheeler, M. Am. Soc. C. E., Engineer.
(4) *Transactions*, Am. Soc. C. E., Vol. LIII.
(5) *Journal*, N. E. W. W. Assoc., June, 1906.
(6) *Engineering News*, November 10th, 1910.
(7) Schuyler's "Reservoirs," 2d Edition.
(8) *Engineering News*, August 24th, 1911, p. 220.
(9) *Engineering News*, October 16th, 1913.
(10) *Proceedings*, Am. Soc. C. E., April, 1914.
(11) *Engineering News*, October 27th, 1910; and *Journal*, N. E. W. W. Assoc., December, 1912.
(12) *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 214.

* 15 ft. above base.

† As designed; actually built only 80 ft. high.

Mr. Herman.

of 0.8 ft. over the top. According to the ordinary method of arch analysis, the stress in the arch at the base would have been 385 lb. per sq. in. Mr. Sherman.

Las Vegas Dam.—In 1910-11, the Agua Pura Company, of Las Vegas, N. Mex., constructed a concrete arched dam, 50 ft. high, to form a storage reservoir. It was designed to be increased in height to 95 ft., subsequently, and it was computed that, even if there should be no bond between the old and new work, and the full water pressure should be transmitted through the resulting joint, the maximum stress in the new work would not exceed 520 lb. per sq. in.

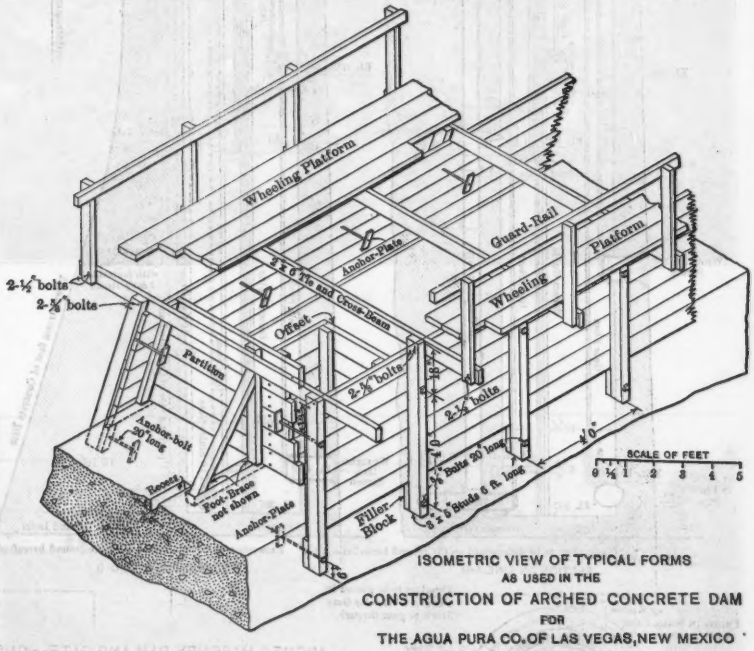


FIG. 18.

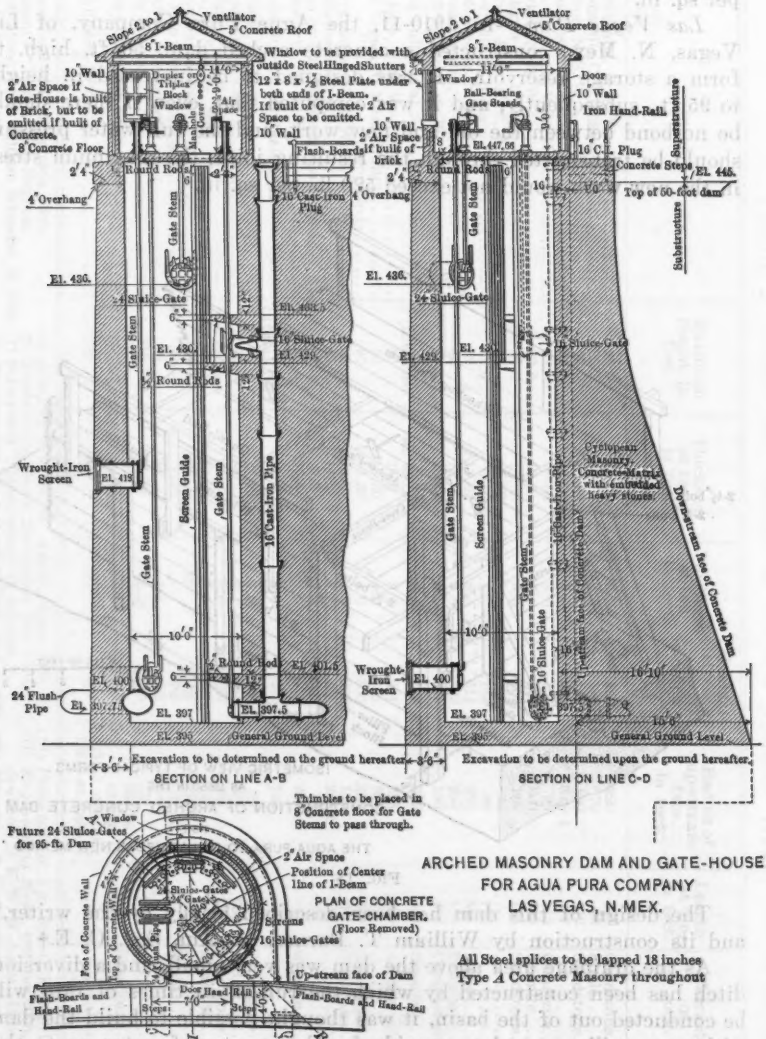
The design of this dam has been described briefly by the writer,* and its construction by William T. Barnes, M. Am. Soc. C. E.†

As the drainage area above the dam was very small, and a diversion ditch has been constructed by which the run-off in times of rain will be conducted out of the basin, it was thought possible to build the dam without a spillway, and to provide for the storage of water up to the level of the crest of the dam. The water is to be supplied to the

* *Engineering News*, Oct. 27th, 1910.

† *Journal*, New England Water Works Assoc., Vol. 26, p. 356.

Mr. ...erman.



ARCHED MASONRY DAM AND GATE-HOUSE FOR AGUA PURA COMPANY LAS VEGAS, N. MEX.

All Steel splices to be lapped 18 inches

Type *A* Concrete Masonry throughout

FIG. 19.

DAM AT LAS VEGAS, N. MEX., FOR AGUA PURA COMPANY.

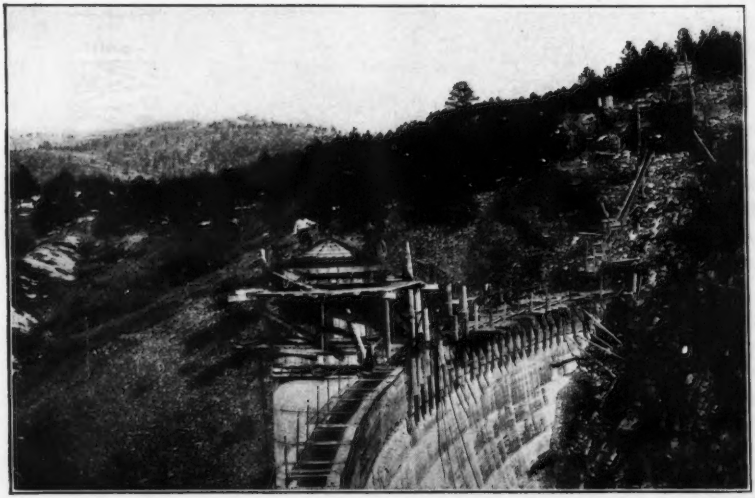


FIG. 20.—TOP OF DAM NEARLY COMPLETED.

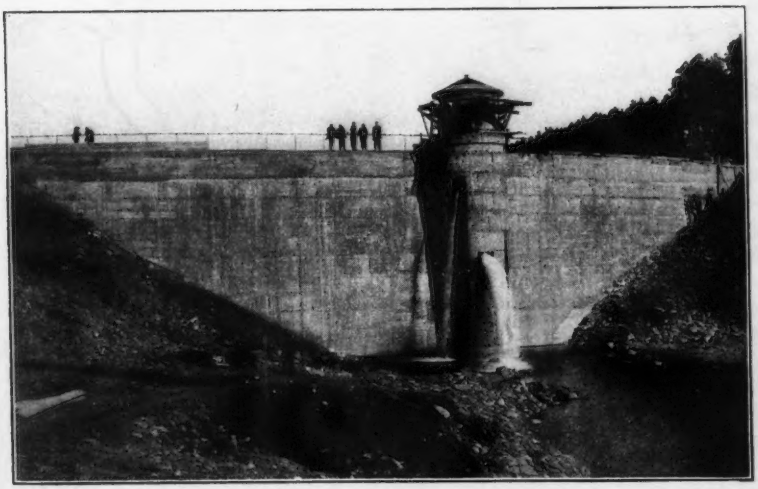
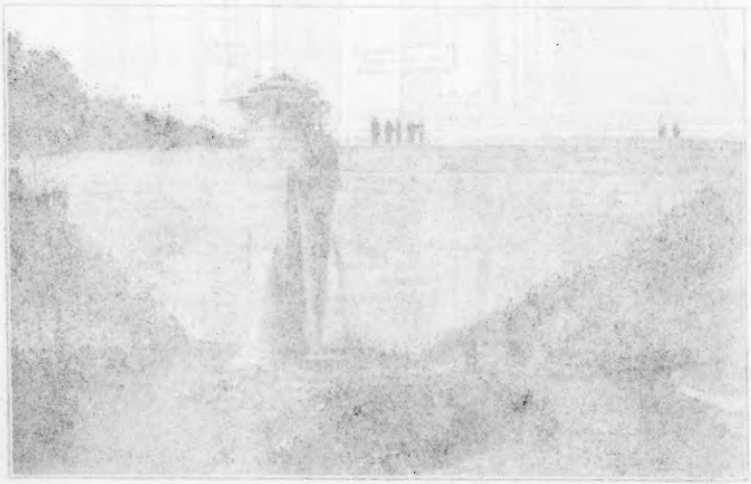
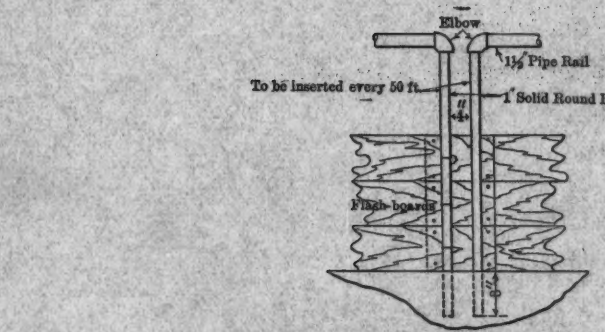
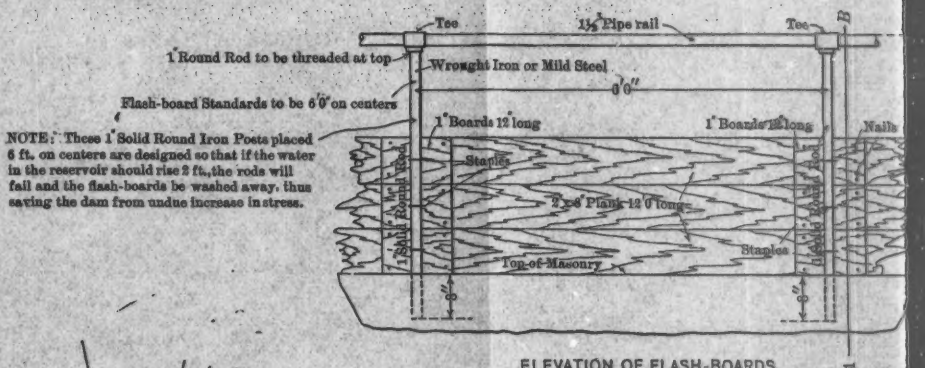


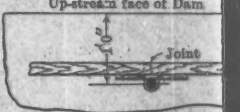
FIG. 21.—UP-STREAM FACE OF COMPLETED DAM.

UNIV. OF MICH.

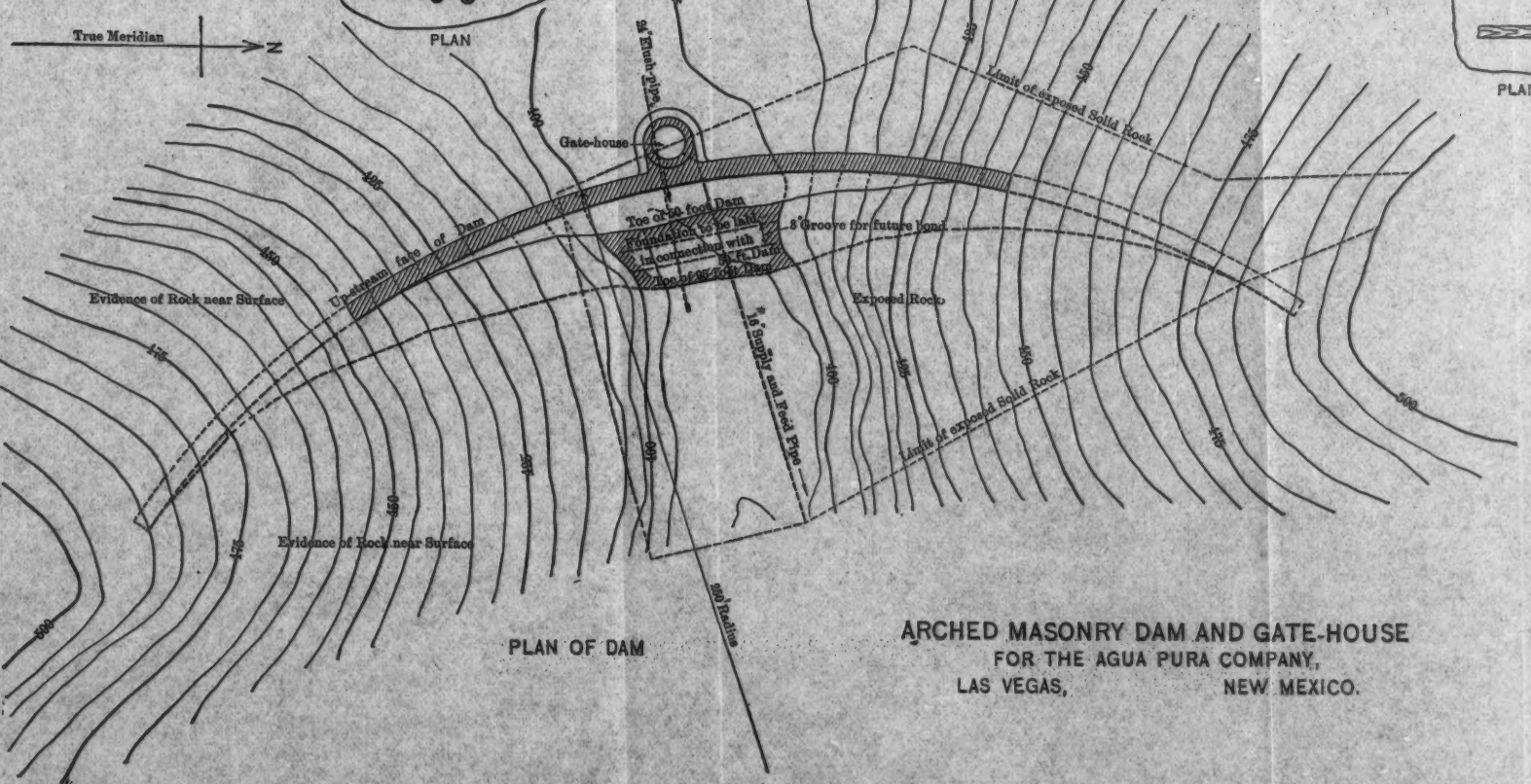








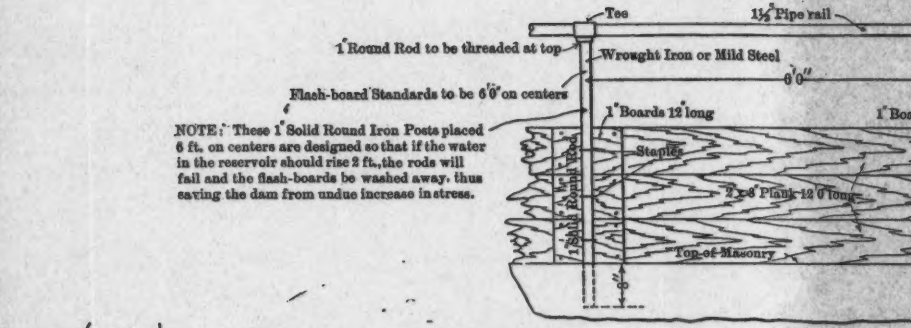
DETA
FLASH-BOARDS



Excavation and
for large Dam sec
out in the bottom
as shown in plan

The lines of this exca
masonry refill will de
developments on the

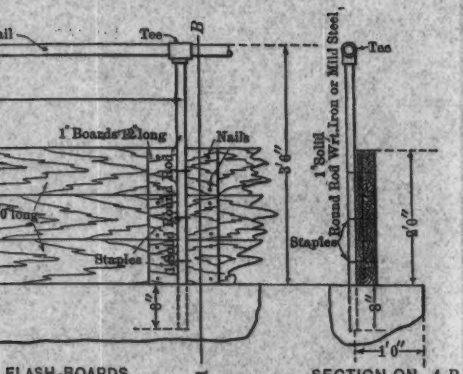
ARCHED MASONRY DAM AND GATE-HOUSE
FOR THE AGUA PURA COMPANY,
LAS VEGAS, NEW MEXICO.



ELEVATION OF FLASH-BOA

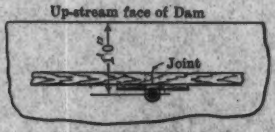
ARCHED MASONRY DAM AND GATE-HOUSE
FOR THE AGUA PURA COMPANY,
LAS VEGAS, NEW MEXICO.

PLATE XXII.
PAPERS, AM. SOC. C. E.
AUGUST, 1914.
SHERMAN ON
CONCRETE ARCH DAM.



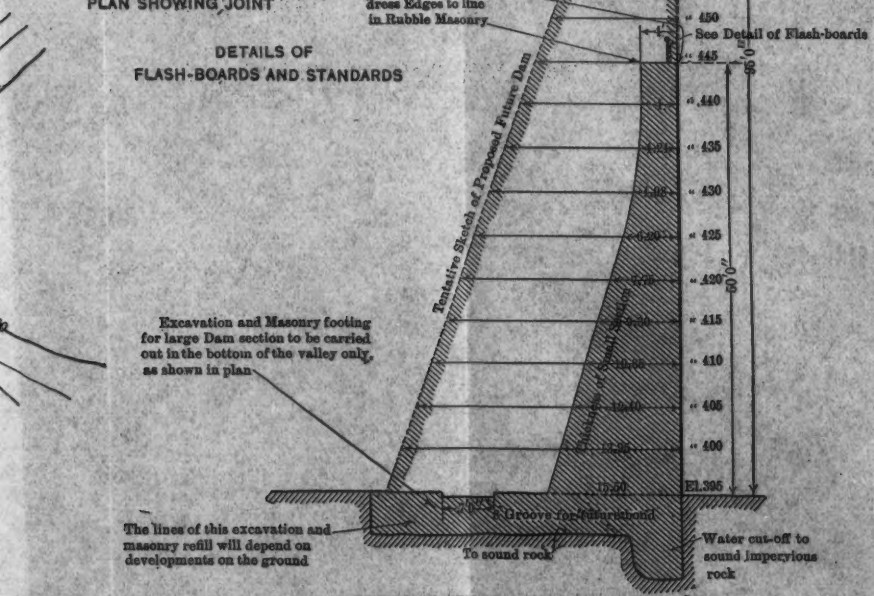
FLASH-BOARDS

SECTION ON *A-B*



PLAN SHOWING JOINT

DETAILS OF
FLASH-BOARDS AND STANDARDS



SECTION THROUGH 50-FT. AND 95-FT. DAM

reservoir partly through a canal and partly through a pipe which also serves as an outlet. It was believed, therefore, that the likelihood of the water rising higher than was desired would be very small indeed, and that, in case of a cloudburst or other emergency causing the water to overtop the dam, no serious damage was to be apprehended. Fig. 18 is an isometric view of the forms as used in building the dam. Mr. Sherman.

In order to make possible the storage of water up to the level of the crest, without allowing waves to pass over, flash-boards have been provided. The flash-board pins were computed to yield and release the boards when the water reached a height of 2 ft. above the crest. The details of the flash-boards and of the hand-rail carried by the pins have been worked out so as to make it impossible for arch action to develop in the railing or boards.

The arrangement of the gate-house, Fig. 19, is believed to be novel, particularly in its compactness and the method adopted for screening the outlets at different depths by the use of a single section of screen.

Plate XXII shows the plan and section of the dam and gate-house, and the appearance of the structure when substantially finished is shown by Figs. 20 and 21.

The methods and costs of construction were stated by Mr. Barnes in the paper before referred to. It may be of interest to note here that the completed structure, including the gate-house, contains 2 703 cu. yd. of concrete, the average cost of which to the contractor was $7.09, and to the water company, $7.86. The excavations for the foundation amounted to 790 cu. yd. of rock and 245 cu. yd. of earth. The average cost of excavation to the contractor was $1.50, and to the water company, $1.43, per cu. yd.

M. M. O'SHAUGHNESSY, M. AM. SOC. C. E. (by letter).—The author's description of the Huacal Dam and the detail of the methods used in its construction, are to be greatly commended on account of the concise and thorough manner in which all elements relating to the project are discussed. Table 1, containing data on dams of this type previously constructed, is very complete, and the reasons advanced for adopting this type at this particular site deserve thoughtful consideration. With a very narrow gorge, foundation rock of excellent quality, and small property values below the site, the risk involved is perhaps justifiable; but with the menace following destruction by earthquake, doubtful characteristics of the foundation, or local weaknesses in the construction due to inferior material or workmanship, which evade the best inspection and supervision, it is very questionable whether it is good policy to take the risk of relying on a structure of such refined proportions, where a great calamity to life and property might follow from a failure. Mr. O'Shaughnessy.

In the case of the Upper Otay Dam, referred to by the author, of about 1 000 000 000-gal. capacity, which lies immediately above the

Mr. O'Shaughnessy.

Lower Otay Dam and Reservoir, of 13 000 000 000-gal. capacity, the writer was always impressed with the consequences to the structures lower down, which might follow a failure of the upper dam from a destructive earthquake. The lower reservoir, however, has a capacity of 13 000 000 000 gal., and is rarely more than half filled, so that there will always be a fair space to take care of the released volume in case of breakage, though a wave action might top the lower dam or crowd the spillway to its capacity.

In dam construction, engineers on the Pacific Coast are often compelled to adopt economies of design which older and richer communities can afford to reject, in other words, they are very often compelled to adopt boldness of design, which, under other conditions, they might not accept.

The cost of the Huacal Dam was $114 000, excluding water rights or reservoir site, and as the capacity is 2 700 acre-ft., the cost was $42 per acre-ft., which is very reasonable for a structure of this type, with a small reservoir basin and a limited water-shed. The concrete work in the structure must have been well done, as the leakage is only 6 000 gal. per 24 hours; and the precautions adopted by Mr. Hawgood to obtain a superior concrete mix are to be commended. In concrete for dam construction and hydraulic work, there is a modern tendency, even in some large works, to be rather slipshod in the methods, precautions, and proportions used, with the inevitable result of after consequences to the structure and damage to the reputation of the engineers.

The spillway provisions seem to be ample, but, even with a little topping of the main structure, injuries should not be feared, on account of the water-cushion provided at the base. A clearer account of the spillway gates would be appreciated. It was not until 5 years ago that a spillway was provided at the Upper Otay; water passed over the crest of that dam, and cut away some material at the foundation, but did not hurt the structure.

A recent inspection of the Lake Spaulding Dam showed a deflection in the center of 3½ in., when full. It will be interesting to note the deflections in the Huacal Dam.

Mr. Hawgood is to be thanked for writing a paper which is so complete and of such great value to those members of the Profession who are interested in structures of this particular type.

Mr. Jorgensen.

L. R. JORGENSEN, ASSOC. M. AM. SOC. C. E. (by letter).—For such a site as that shown on Fig. 5, there can be no doubt that an arch dam is the safest and cheapest type that could be put in. The (approx.) box canyon in which the 100-ft. dam is located is only a little more than 100 ft. wide at the crest. This narrowness of the canyon allows the use of a short up-stream radius, which is the main requirement for high economy in the arch dam. The axial

Mr. Jorgensen.

stresses chosen by Mr. Hawgood are very conservative, as they ought to be for a structure having such fairly slender proportions as the Huacal Dam. The writer does not quite see the use of the vertical reinforcement in an arch dam of the Huacal type, but, after all, this steel is a small item.

It would be better if engineers in general referred to dam faces as the up- and down-stream faces instead of front and rear faces, as this latter definition is not as clear as the former and is often used inconsistently.

Mr. Hawgood has built a very good substantial structure and need not make any excuses for putting in an arch dam. An engineer who would recommend any other type for the site in question would have a hard time finding enough excuses to justify such a decision.

With the exception of the formula giving the down-stream deflection of the crown of the dam under load, all other formulas used in arch dam design, while they are not absolutely accurate, are as accurate as those ordinarily used in the field of practical mechanics.

In dam design the load is definitely known; it is not, however, definitely known, where this load applies itself, unless the up-stream face is provided with a fairly water-tight skin, such as a plaster put on with a concrete gun. This is one reason why dams having a slender section ought not to be stressed (unit stress) as high as arch dams having a heavier section.

In the formula for the deflection of the crown the modulus of elasticity of the concrete appears. Some value for this has to be substituted in the formula, and that is where the biggest assumption in dam design is apt to be made.

To know, beforehand, what the deflection of the dam, when loaded, is going to be, is of small real importance, and if it turns out to be different from the calculated deflection, it is thereby indicated that the modulus of elasticity of the mass of concrete is different from the assumed one. The modulus of elasticity appears in other formulas, but in such a way that it cancels, and, therefore, no assumptions as to its actual value need be made.

According to the writer's view, an arch dam can be calculated as accurately as most structures, and when it is considered how far from its crushing strength the material is ordinarily working, such a structure should be readily accepted by all engineers as the safest, as well as the cheapest, that can be built in such a place as that described by Mr. Hawgood.

The automatic overflow gates provided for the Huacal Dam are very simple and evidently effective, and undoubtedly will give good satisfaction.

Mr. Mensch.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—The Huacal Dam and the many others mentioned in this paper should convince the most skeptical engineer that arched dams with cross-sections much lighter than those of gravity dams, have a fair factor of safety. None of these structures ever failed or showed larger or more numerous cracks than those found in gravity dams. Arched dams, being considerably cheaper than gravity dams, would be often used in place of earthen dams but for the fact that engineers have no confidence in the rather primitive, although evidently adequate, method by which the pioneer builders determined the cross-section of these structures. Engineers are afraid that some hidden defects exist in these dams, for which they cannot satisfactorily account by theory and for which they do not care to become responsible.

The objections to the assumption that arched dams act like cylinders subjected to external pressure are the following:

First.—These dams are arches fixed at the abutments, and like other arches are subjected to stresses due to the shortening of the arch, change of temperature, and shrinkage and swelling of the concrete. When the reservoir is filled, the swelling of the concrete and the fall of temperature from the original temperature of setting counteract each other to a great extent, and only in case of the reservoir being empty there may exist the most unfavorable condition of a fall of temperature and shrinkage of concrete acting together with the tendency to produce cracks. It can be shown that these stresses are of importance only in the upper half of the dam.

Second.—It is evident that these dams act partly as gravity structures, and the unknown extent to which they act as such is the main objection of thoughtful engineers to this type of dam. In the discussion* on the paper by G. N. Houston, M. Am. Soc. C. E., on the Halligan Dam, the writer has shown that the greatest portion of the water pressure is resisted by gravity action. He is now obliged to confess that he overlooked an important point. He assumed that the dam acted as a gravity structure entirely within the limit of ordinary beam action, following Hooke's law. This is not the case in masonry dams after the resultant of forces strikes the base outside the middle-third, or in a reinforced structure after the stresses in the steel bars reach the elastic limit. In both cases after cantilever action is nearly exhausted we have the condition of the dam being rather hinged at the bottom than immovably fixed, as formerly assumed by the writer, and if this condition is assumed, theory gives results which agree exceedingly well with practice.

It is an acknowledged and very plausible law of Nature that, among all possible deformations which a structure subjected to exterior

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 185.

forces can perform, deformation will take place, which will make the work of the internal forces a minimum. Mr. Mensch.

It can be proven that the work of compressing the structure as an arch (as long as the water level remains not higher than about 30 to 40% of the total depth of the water above the base of the dam) is much greater than that necessary to deflect the structure as a cantilever, hence gravity action must be the preponderant influence by which the stresses at the bottom of the dam are governed when the reservoir is about half full.

If, instead of an arch, we assume a complete cylinder like a water tank, it follows that the walls of the tank cannot move, even if set on a frictionless base, before the cantilever action is exhausted. The futility of all devices which were ever patented to disconnect walls of tanks from the base is thereby shown.

It can be shown that in all arched dams, with the possible exception of the Zola Dam, the lower portion of the structure of a height of about 5% of the total depth of water is strained far beyond anything possible in a gravity structure, yet no harm can come to the dam, because when gravity action ceases, arch action takes its place, and while probably every gravity dam would fail should the water rise to twice the height of the dam, some of the arched dams mentioned by Mr. Hawgood have a factor of safety of three.

The author thinks that both the rock at the base and at the abutments of the dam moves in accordance with the dam, and hints that this may explain his success. Any rock which would allow such movements would be entirely unsuitable for an important arch structure, and to assume that a very much larger mass of rock than the mass of the dam allows anything but infinitesimal deformations, is out of the question.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HOWARD ELMER ARTHUR, M. Am. Soc. C. E.*

DIED APRIL 19TH, 1914.

Howard Elmer Arthur was born at Warren, Ohio, on January 26th, 1872. He was educated in the public schools of Washington, D. C., and studied architecture for one year in the office of Messrs. Smithmeyer and Pelz of that city.

In August, 1891, Mr. Arthur came to New York City and entered the office of the Jackson Architectural Iron Works, where, for 5 years, he was engaged in detailing and checking the steelwork for office buildings, etc., erected by that firm. In August, 1896, he went to Paterson, N. J., where he was employed by the Passaic Rolling Mill Company, detailing and checking trusses, plate girders, etc.

In April, 1897, Mr. Arthur returned to New York City, where he was engaged, until December, 1898, with the firms of Cooper and Wigand and Lewinson and Just, in detailing and checking, and in charge of construction work. He resigned his position with the latter firm to go with the Union Bridge Company, at Athens, Pa., on detailing and checking and in charge of work. Slackness of work at the Union Bridge Company's plant caused Mr. Arthur to return to his former position with Messrs. Lewinson and Just in September, 1899, where he remained until March, 1900, when he went to St. Louis, Mo., with the Koken Iron Works, as Assistant Engineer. While in this position, he had charge of the construction of the Frisco Building, the Mississippi State Capitol Building, mining and mill buildings, tanks, etc. In 1902, Mr. Arthur was transferred to the Contracting Department of the American Bridge Company as Contracting Agent, and was also engaged on designing and estimating the cost of steel structures.

In September, 1904, he went to Dallas, Tex., as Chief Engineer of the Mosher Manufacturing Company, in which position he designed and estimated the costs for steel construction for various shops and buildings at Dallas and Houston, churches, highway bridges, tanks, domes, etc., as well as the Grand Stand at the State Fair Grounds at Dallas, which latter has been described by Mr. Arthur.†

In October, 1911, in order that his child, who was seriously ill, might receive proper medical treatment, Mr. Arthur returned to New

* Memoir prepared by the Secretary from information on file at the Society House.

† *Engineering News*, Vol. LX, 1908, p. 208.

York City. He entered the office of Olaf Hoff, M. Am. Soc. C. E., and, under Mr. Hoff's direction, made the drawings and calculations for the City Art Bridge from Fortieth to Forty-second Streets, connecting to the Grand Central Station, and various other constructions. He remained with Mr. Hoff until March, 1912, when, owing to failing health, he was compelled to resign. He spent the next two years in Vermont, Tryon, N. C., and the Catskill Mountains, in a vain endeavor to improve his health, but died of tuberculosis at Big Hollow, N. Y., on April 19th, 1914.

On March 17th, 1905, Mr. Arthur was married to Miss Nettie Thomas, who survives him.

Mr. Arthur was elected a Member of the American Society of Civil Engineers on November 8th, 1909.

JAMES JOSEPH FERRIS, M. Am. Soc. C. E.*

DIED MAY 15TH, 1914.

James Joseph Ferris, the son of John and Mary (Harrington) Ferris, was born in Listowel, County Kerry, Ireland, on September 17th, 1860. When he was five years of age, his family came to the United States and settled at Carmel, Putnam County, New York, where they lived for six years, until ill-health and business reverses caused their return to Ireland. Mr. Ferris' early education, started at Carmel, was completed in Ireland, and, after leaving school, he helped his father, who was the manager of a large estate.

After a few years at farm work and at the age of twenty, Mr. Ferris returned to the United States and settled in Jersey City, N. J. He entered the employ of D. S. Cofrode and Company, as Timekeeper, and, subsequently, as Foreman, on important bridge work and other heavy railroad construction in Maryland, Pennsylvania, and Virginia.

Early in 1886, during the settlement of the estate of this firm, he was made Superintendent by the executors, and was in charge of all the work being done by the firm. When a re-organization was effected in 1888, under the firm name of Sandford and Stillman Company, Mr. Ferris was appointed General Superintendent, and held this position until 1902, when the firm was re-organized and became known as The F. M. Stillman Company, Mr. Ferris remaining as General Superintendent. In 1911, the firm was again re-organized under the name of Stillman-Delehanty-Ferris Company, Engineers and Contractors, and Mr. Ferris became Vice-President and Chief Engineer, in charge of all engineering and outside work. About a year ago (1913), he became President of the Company.

* Memoir prepared by Louis Chevalier, Assoc. M. Am. Soc. C. E.

Mr. Ferris' engineering education was obtained at Cooper Union Institute in New York City, and through constant home study. He was a keen observer, possessed a wonderful memory, and qualities for thoroughness, ready debate and argument, and was of a most genial and happy nature. Accustomed to directing the labor of large numbers of men, he had fine tact, most courteous manners, and, in a very high degree, the power to command loyal service and friendship from others.

He was in charge of various parts of the construction of the Pennsylvania Railroad Company's train-shed and office building, and the ferry slips and sheds at Jersey City, and directed the work of the grade crossing elimination for the same railroad through that place. He was also in charge of the construction of the roundhouse and yards of the New York, Susquehanna and Western Railroad, at Shadyside, and of various other work about New York Harbor, notably the sinking of the shafts at Washington Street and at Exchange Place, in Jersey City for the tube elevators for the Hudson and Manhattan Railroad Company, and the construction of the Hackensack River Turnpike Bridge.

In June, 1913, Jersey City adopted a Commission form of Government, and Mr. Ferris was signally honored by being elected a member of the first Board to govern its civic affairs. He was selected to take charge of the Department of Streets and Public Improvements, and during the past year he had brought his Department to a high state of efficiency, his most notable achievement being a thorough cleaning up of the City's water supply on the Boonton water-shed.

Mr. Ferris died suddenly on May 15th, 1914. He was a thorough and painstaking engineer, an exceptionally devoted student, was esteemed and admired by all who knew him, a faithful friend, a devoted father, and a good citizen.

In 1888, he was married to Miss Bridget Ryan, of Baltimore, Md., who, with five sons and three daughters, survives him.

Mr. Ferris was elected an Associate of the American Society of Civil Engineers on July 10th, 1907, and a Member on September 3d, 1912.

WEBSTER GAZLAY, M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1914.

Webster Gazlay, was born in Louisville, Ky., on February 17th, 1862, and given the name of "Webster" in recognition of his father's admiration for the great statesman. His surname has come through mutations from Gazeley, Gazelle, Geasley, and Gaselee, of early Eng-

* Memoir prepared by J. M. Johnson, M. Am. Soc. C. E.

land. For the past 400 years, Judges of the name of Gaselee have sat on the King's Bench, in England, and their decisions are still quoted in English books of law. A Judge Gaselee was Dickens' prototype, it is said, for "Judge Stareleigh" in Pickwick's famous breach of promise suit.

Mr. Gazlay's mother was a Wheeler, of Virginia ancestry, Chaplines and Claibornes blending. On her maternal side, John Randolph, of Roanoke, was a kinsman.

In 1715 (Old Style) the English progenitor of the family in America settled in the State of New York, and his son, John Gazlay, was a member of the Continental Congress. Webster Gazlay's father, Addison M. Gazlay, was born in the valley of the Unadilla River, in Central New York, went to Kentucky in his early manhood, and for many years was a leading commercial lawyer at the Louisville bar.

In spite of the marked inclination in the family to the pursuit of law, Mr. Gazlay early manifested such a love and aptitude for mathematics, that he easily stood at the head of his classes in such studies. This passion for mathematical investigation, undoubtedly influenced his choice of a profession and determined the precision of thought and act and the exactness in expression and language, that were matters of comment and banter among his intimate friends.

He was graduated from the Louisville Male High School in 1881, and, the death of his father preventing the further prosecution of his technical studies, he accepted a position as Draftsman in the office of the Chief Engineer of the Louisville and Nashville Railroad, where he remained until May, 1883.

After leaving the Louisville and Nashville Railroad Company, he was appointed Assistant to the late Charles Hermany, Past-President, Am. Soc. C. E., and was engaged on the construction of water-works at Frankfort, Ky., and on surveys, plans, etc., for a proposed reservoir for Nashville, Tenn.

On the completion of these projects, he became Assistant Engineer on the Louisville Southern Railroad (now a part of the Southern Railway System), and was in charge of a residency during the construction of that road.

In 1888, Mr. Gazlay accepted a position, as Assistant Engineer, with the Louisville Water Company, later becoming Assistant to the Chief Engineer, during the construction of a new pumping station and an extensive system of filters; he remained in this position until 1905, resigning at that time to become the Vice-President and Engineer of the National Concrete Construction Company, a contracting firm with its principal office at Louisville, Ky.

In 1906, he again returned to the Louisville Water Company as Associate Engineer, and retained this position until the completion of the filter plant in the Fall of 1909. In accepting his resignation,

the Water Board passed the following resolution to be recorded in its Minutes:

"To his intelligence in conception, vigor in execution, and tireless energy in the carrying on of the work, is due, in large measure, the successful completion of the great enterprise to which he has devoted himself."

Mr. Gazlay was married on November 7th, 1906, to Miss Lida Hampton, of Louisville, Ky., who, with three children, Webster Gazlay, Jr., Lida Hampton Gazlay, and Sallie Josephine Gazlay, survives him.

Mr. Gazlay was a man of high ideals and thoroughly in love with his Profession. All his concepts were strong and clear, touched in a great degree with that light and leading which humanity is wont to term genius. His standard condemned anything less than the wholly clean way of honesty and truth, and permitted no compromise where the facts were clear and plain. To his many friends, his untimely passing away is a source of great sorrow, but they have the consolation of knowing that it has been their privilege to work and walk with a man fashioned well and truly in the image of his Maker.

He was one of the original members of the Engineers and Architects Club of Louisville, Ky., and was always placed on committees where important and efficient work was demanded.

Mr. Gazlay was elected a Member of the American Society of Civil Engineers on June 7th, 1905.

WILLIAM AUGUST HUNICKE, M. Am. Soc. C. E.*

DIED MARCH 9TH, 1914.

William August Hunicke was born at St. Louis, Mo., on February 23d, 1875. He attended the public schools until 1889, when he was sent to Germany where he studied for three years. On his return to St. Louis, he entered Washington University, from which he was graduated in 1897, with the degree of B. S. in Civil Engineering.

His first employment was with Robert Moore, Past-President, Am. Soc. C. E., and the late J. B. Johnson, M. Am. Soc. C. E., after which he spent five years with the Mexican Central Railroad, in charge of the location and construction of various branch lines of that Company. Returning to St. Louis, he was placed in charge of the Washington University buildings and the laying out and improvement of the University grounds.

Mr. Hunicke was Assistant Engineer of the St. Louis Terminal Railroad during the time that Mr. Taylor was Chief Engineer, and in that capacity had charge of the relaying of the track in the tunnel,

* Memoir prepared by Leo C. Dziatzko, Esq., Webster Groves, Mo.

which work had to be accomplished without interference with the traffic. His next work was the location and construction of the extension of the railroads owned by the Sligo Furnace Company in Missouri.

From 1909 to 1911, he was Chief Engineer and Superintendent of Construction of the Apalachicola Northern Railroad, locating and building that road from Apalachicola to Jacksonville, Fla.

From 1911 to 1913, he was Superintendent of Construction for the English Syndicate in Cuba on various extensions of its roads, returning to St. Louis in August, 1913, to become Assistant Operating Engineer of the Missouri Pacific Railroad, which position he held at the time of his death.

Mr. Hunicke was a member of the Engineers' Club of St. Louis, the Railway Club of St. Louis, and the Missouri Athletic Club where he made his home, enjoying its comforts and environments. He was one of those who perished in the fire which destroyed the building on March 9th, 1914.

He was one of the best posted authorities on railroad location, construction, and valuation, and acted as Consulting Engineer on valuation work for a number of the smaller railroads of the South. Beloved by his friends, his was a life well spent in constructive work, and those who knew him well cherish his memory.

Mr. Hunicke was elected an Associate Member of the American Society of Civil Engineers on March 2d, 1904, and a Member on January 7th, 1913.

THOMAS HUMRICKHOUSE JOHNSON, M. Am. Soc. C. E.*

DIED APRIL 16TH, 1914.

Thomas Humrickhouse Johnson was born at Coshocton, Ohio, on January 12th, 1841. His parents, William Kerr Johnson and Elizabeth (Humrickhouse) Johnson became residents of Coshocton in their youth, during the first quarter of the last century.

William Kerr Johnson was born in Dungannon, County Tyrone, Ireland, of Scotch-Irish parentage, in 1809, and, while yet a boy, came to America with his brothers, residing first in Baltimore and moving West, in 1821, to the Village of Coshocton.

Elizabeth Humrickhouse was a descendant of a Prussian family which emigrated to America from the Duchy of Hesse Darmstadt in the early part of the 18th century, settling first at Bethlehem and Germantown, Pa., and at Hagerstown, Md. From Hagerstown, Peter Humrickhouse, the father of Elizabeth, moved West, finally settling in Coshocton.

* Memoir prepared by J. P. Snow, J. C. Bland, and R. Montfort, Members, Am. Soc. C. E.

Mr. Johnson's early education was under the tutelage of the Rev. William E. Hunt, then pastor of the Coshocton Presbyterian Church, under whom he was fitted for college.

In 1858 he entered old Jefferson College, at Canonsburg, Pa., and after three years was graduated at the age of twenty years, with the degree of Bachelor of Arts, in the class of 1861. On account of his unusual proficiency in mathematics, the degree of Master of Arts was conferred on him in 1866 by his Alma Mater, and in 1911 the degree of Doctor of Science by Washington and Jefferson University, the successor to the former colleges of Washington and Jefferson.

On graduation Mr. Johnson entered the employ of the Pittsburgh and Steubenville Railroad, of which his father was a director, as Rodman; and in 1863 he became Assistant Engineer in the construction of the Steubenville Bridge over the Ohio River. From 1867 to 1869 he served as Assistant Engineer on general work on the Pittsburgh, Cincinnati and Chicago Railway, from 1869 to 1871 on the construction of the Chartiers Railway, and in 1871 was transferred to Richmond, Ind., where he was engaged in the erection of the passenger station at that place until May 31st, 1872. From that date until March 1st, 1873, he served the Company in the general work along its lines, and then began the erection of the depot at Columbus, Ohio. On the completion of this work, in 1875, he was again transferred to Indiana, and throughout that year was in charge of construction of the Vincennes Engine House and Richmond Shops.

In the latter part of 1875 he severed his connection with the railroad and opened an office in Columbus, Ohio, for architectural work, and thereafter until March, 1883, was engaged as Chief Engineer in the construction of the present State Capitol at Indianapolis.

In 1883 he again entered the service of the Pittsburgh, Cincinnati, Chicago and St. Louis Railway, as Principal Assistant Engineer, which position he occupied until 1896, when, on the death of M. J. Becker, Past-President, Am. Soc. C. E., he was appointed to the position of Chief Engineer of the Southwest System. In 1901 he became Consulting Engineer for all Lines West of Pittsburgh, a position which he held until January, 1911, when he reached the age (70 years) of automatic retirement.

The services of Mr. Johnson were considered so valuable by the management of the Pennsylvania Railroad that the positions of Chief Engineer of the Pittsburgh, Chartiers and Youghiogheny Railroad, the Chartiers Southern Railway, and Special Consultant for the Pennsylvania Lines were created for him. This procedure, which is without precedent in the history of the Pennsylvania System, constitutes a striking tribute to his capacity and worth. It is a tradition among the Pennsylvania men that no subject was ever broached to him which failed to elicit information to the point and offhand.

In addition to his regular duties, his advice and counsel were frequently sought, and highly valued in cases where expert judgment

was required in difficult constructions, and in cases of adjustment between parties with opposing interests.

Mr. Johnson found especial enjoyment in association, not only with his fellow-engineers, but with all men of learning, and to that end he accepted many invitations to membership in scientific bodies. Besides membership in the American Society of Civil Engineers, he was a member of the American Railway Engineering Association; the American Society for the Advancement of Science; the American Society for Testing Materials; the American Forestry Association, and the Engineers Society of Western Pennsylvania. He was a member of the Rail Committee of the American Railway Engineering Association, of the Special Committee on Valuation of Public Utilities, of the American Society of Civil Engineers, and of the Rail Committee of the Pennsylvania System.

Of several of these societies he has at times been on the board of officers, and has served them on committees and as a delegate to various congresses. The Engineers Society of Western Pennsylvania is especially indebted to him, not only as an active officer, but as an adviser and mentor in times of stress, and as one ever ready to carry the burden of unusual activities and grave responsibilities whenever the needs of the society called him.

He was a valued contributor to the publications of many of the associations of which he was a member. To the American Society of Civil Engineers he contributed a notable paper "On the Strength of Columns; Discussing the Experiments which have been Accumulated, and Proposing New Formulas,"* and discussions on the cause and prevention of decay of building stone; a formula for the strength of columns; Progress report of Special Committee on Steel Columns and Struts; railway bridge designing; stresses in railway bridges on curves; and ventilation of tunnels. His paper and discussion on the Strength of Columns brought out the famous straight-line formula which he demonstrated to be the tangent and rational extension of Euler's curve. His discussion on the ventilation of tunnels was the result of his translation of a work by the Italian Government on tunnel ventilation, an abstrusely mathematical treatise which Mr. Johnson translated with the aid of an Italian dictionary and grammar in combination with his extraordinary mathematical ability.

He was 61 years of age when he translated this work, and when one of his engineer friends remarked on his industry and courage in undertaking such a task, he pointed to a page of the work and asked, "what constitutes the principal part of that page?" The friend answered, "mathematical expressions." "Yes," said Mr. Johnson, "a universal language."

In 1868 Mr. Johnson married Miss Martha E. Patterson, of Steubenville, Ohio, who survives him with two daughters, Misses

* *Transactions*, Am. Soc. C. E., Vol. XV, p. 517.

Bessie and Margaret Johnson, and one son, William K. Johnson, who is an attorney at law in Pittsburgh, Pa.

As a man, Mr. Johnson's notable characteristics were: perfect sincerity, single-mindedness, consideration for others, kindliness, charity, and a constant readiness to help others; and his utter uncomplaining, even in the hour of death. His vast fund of well-ordered knowledge was always at any one's command; and the younger men went to him for advice, counsel, and help as freely as a child to its father.

John Henry Newman says:

"The true gentleman carefully avoids whatever may cause a jar or a jolt in the minds of those with whom he is cast; all clashing of opinion or collision of feeling, all restraint, or suspicion, or gloom, or resentment; his great concern being to make every one at their ease and at home. He has eyes on all his company; he is tender toward the bashful, gentle toward the distant, and merciful toward the absurd; he guards against unreasonable allusions, or topics that may irritate; he is seldom prominent in conversation and never wearisome. He makes light of favors when he does them and seems to be receiving when he is conferring. He never speaks of himself except when compelled; never defends himself by a mere retort; he has no ears for slander or gossip; is scrupulous in imputing motives to those who interfere with him, and interprets everything for the best. He is never mean or little in his disputes, never takes unfair advantage; never makes personalities or sharp sayings for arguments or insinuates evil which he does not say out. * * *

"He is patient, forbearing, and resigned, on philosophical principles; he submits to pain because it is inevitable; to bereavement because it is irreparable, and to death because it is his destiny."

Such a man was Thomas H. Johnson.

Mr. Johnson was elected a Member of the American Society of Civil Engineers on September 5th, 1877, and served as a Director during the years 1900, 1901, and 1902.

NATHANIEL TURNER, M. Am. Soc. C. E.*

DIED JANUARY 19TH, 1914.

Nathaniel Turner was born in Scituate, Mass., on March 27th, 1861, and was educated at the public schools of that place, and at Bingham Academy.

Leaving home at the age of nineteen, Mr. Turner went West and began work as a Rodman on the location of the Milwaukee, Lake Shore and Western Railroad, in Wisconsin, in 1880. In 1881, he was made Transitman on the same work.

In 1882, Mr. Turner left the employ of the Milwaukee, Lake Shore and Western Railroad and went to Mexico where he was appointed

*Memoir prepared by the Secretary from information on file at the Society House.

Levelman on the line of the Mexican National Railway then being constructed between Laredo and the City of Mexico. He remained with this Company until 1888, having served as Transitman, Resident Engineer, and Bridge Engineer, respectively. In 1884, he was employed on trigonometrical surveys of large land areas in the State of Coahuila, Mexico.

In 1888, Mr. Turner was appointed Bridge Engineer and in 1889 Locating Engineer on the Monterey and Mexican Gulf Railway, then building between Monterey and Tampico, under Mr. William H. Wentworth, Chief Engineer. In 1890, he was made Principal Assistant Engineer in charge of construction, remaining in that position until 1892, when he resigned to become Chief Engineer of the Matehuala Porvenir Railway. He also served as Locating Engineer on the Gulf and Pacific Railway. In 1893, he was appointed Chief Engineer of the Monterey and Mexican Gulf Railway, in which position he remained until 1897.

From 1897 to 1901, Mr. Turner was engaged in the private practice of engineering. In the latter year he was appointed Superintendent of Construction of the Monterey Steel Works, a $10 000 000 plant. On the completion of this work in 1904, he again returned to private practice for a year, at which time he went to Matehuala, San Luis Potosi, as General Superintendent of the Matehuala Smelter and Water-Works.

On account of ill-health, Mr. Turner resigned this position in 1908, and retired from active business. He owned a ranch in Tamaulipas and oil investments near Tampico, and devoted his time to these interests, making his home in Monterey. He died of heart disease on January 19th, 1914, and was buried in the Carmen Cemetery, at Monterey.

In 1885, Mr. Turner was married to Elizabeth Luella Donaldson, of Rock Island, Ill., who, with one daughter, survives him. He was one of the best known and best loved Americans in Northern Mexico, and had served as President of the American Colony in Monterey.

Mr. Turner was elected a Member of the American Society of Civil Engineers on March 6th, 1895.

GEORGE JOHN COUCHOT, Assoc. M. Am. Soc. C. E.*

DIED MAY 3D, 1914.

George John Couchot was born in Paris, France, on August 23d, 1874. In October, 1887, he came to the United States, going to Oakland, Cal., where he attended the High School.

After a course in an Engineering School, Mr. Couchot, in October,

* Memoir prepared by the Secretary from material on file at the Society House, supplemented by information supplied by Maurice C. Couchot, M. Am. Soc. C. E.

1892, was engaged by the City Engineer of San Luis Obispo, Cal., on transit work. In March, 1893, he went to the Fulton Iron Works, in San Francisco, as Mechanical Draftsman, remaining in that position until August, 1894. From April to December, 1895, he was employed by the State Harbor Commissioners of California, leaving to go with the Western Sugar Refining Company, with which Company he remained until December, 1896.

From January to April, 1897, Mr. Couchot was employed as Draftsman in the office of Mr. J. C. H. Stut, Mechanical Engineer, and was engaged on the design and construction of the steel frame building for the Western Beet Sugar Factory, at Watsonville, Cal., until August of the same year. He then entered the service of the Fulton Iron Works as Mechanical Draftsman, which position he held until August, 1898.

From October, 1898, to March, 1900, Mr. Couchot was employed as Mechanical and Ship-Building Draftsman by the Union Iron Works, of San Francisco, but resigned to accept again a position with the Fulton Iron Works. In March, 1901, he was appointed Assistant Engineer of the Western Sugar Refining Company, in charge of the design and construction of all that Company's improvements, buildings, machinery, etc.

In February, 1905, he was employed, on the design of steel structures, by Maurice C. Couchot, M. Am. Soc. C. E., Consulting Engineer, with whom he remained until June, when he was appointed Draftsman in the office of the City Engineer of San Francisco. Mr. Couchot retained this position until September, 1907, when he became connected with the firm of Couchot and Thurston, Consulting Engineers, on the design of structures. In August, 1908, he returned to the City Engineer's office where he remained as Chief Draftsman and Assistant Engineer in charge of some of the sewer design and construction, until his death on May 3d, 1914, after an operation for appendicitis.

Mr. Couchot was held in high esteem by all who came in contact with him. His wife and a young son survive him.

Mr. Couchot was elected an Associate Member of the American Society of Civil Engineers, on May 3d, 1910.

ALBERTO DE LA TORRE, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 5TH, 1912.

Alberto de la Torre, the son of Demetrio de la Torre and Maria Josefa Umaña, was born in Tocaima, Colombia, on August 6th, 1876. He received his elementary education in his native country, and in June, 1889, was sent to the United States to begin the study of Civil

*Memoir prepared by Mr. Diodoro Sánchez, President of the Colombian Society of Engineers, Bogota, Colombia, supplemented by information on file at the Society House.

Engineering at Rensselaer Polytechnic Institute. He was graduated in 1897, with the degree of Civil Engineer, and returned to Colombia to devote himself to the practice of his profession.

Mr. de la Torre was engaged in private practice in Bogota from March, 1897, to August, 1898, his work consisting of surveying and general engineering. In September, 1898, he was appointed Assistant Engineer of Maintenance of Way on the Ferrocarril de la Sabana, but resigned this position in December, 1898, and engaged again in surveying and general engineering practice.

In April, 1899, Mr. de la Torre accepted the position of Assistant Engineer on the location of the Girardot Railway (now the Colombian National Railway), and remained in this position until the completion of the location to San Joaquin, in July, 1899, when he was appointed Assistant Engineer, under Mr. T. B. Nowell, on the location of the Dorada Railway Extension to the Port of Cambao.

From October, 1899, to April, 1901, he was again engaged in the private practice of surveying and architectural work, and from April to October, 1901, was employed temporarily as Engineer of Way Works on the Dorada Railway.

He was employed as Assistant Engineer on the location and construction of the Girardot Railway from October, 1901, to May, 1904, and, during this time, also had charge of the construction of several private residences, and made the surveys for a cart road on a private estate.

From May, 1904, to July, 1905, Mr. de la Torre was engaged as Engineer and partner of the contractor, on the construction of 2 km. of the Girardot Railway, and from July to December, 1905, he was employed as Assistant Engineer on surveys and estimates for the reconstruction of the Cambao Road. He was afterward appointed by the Government of Colombia to direct the work on the Central Road, between the Puente del Común and Chocontá. In 1908, he again entered the employ of the Girardot Railway Company, being engaged on the construction of the last sections between Zipacón and Facatativá. He was then appointed Resident Engineer in charge of the last section from Girardot to Juntas de Apulo, which position he held until July 1st, 1912.

Mr. de la Torre was a member of the Colombian Society of Engineers, and the following is a translation of part of an obituary notice which appeared in the November, 1912, issue of the *Anales de Ingenieria*, the official organ of that Society:

"His constant labor for a dozen years, and his continual battle with the inclemencies of our tropical climate, destroyed his energy and weakened his physique to the extent of bereaving his country and his friends of a noble young life which had already rendered distinguished professional services."

Mr. de la Torre was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1906.

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"A METHOD OF DETERMINING STORM-WATER RUN-OFF." CHARLES B. BUERGER. (To be presented Oct. 7th, 1914.)

"THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS." S. W. BOWEN. (To be presented Oct. 21st, 1914.)

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A METHOD OF DETERMINING STORM-WATER RUN-OFF." Charles R. Rockey. (To be presented Oct. 7th, 1914.)

THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS." R. W. Stowe. (To be presented Oct. 21st, 1914.)

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THE CLARIFICATION OF SEWAGE BY FINE SCREENS." Rudolph Allen. (To be presented Oct. 21st, 1914.)

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"Hurricane Dam, Somora, Mexico." H. Hargood. ..... Apr., May, June, July, Aug., Sept., Oct., Nov., Dec., 1913; Jan., Feb., Mar., Apr., May, June, July, Aug., Sept., Oct., Nov., Dec., 1914

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PAPERS AND DISCUSSIONS

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT.

By H. T. Cory, M. Am. Soc. C. E.

To be Presented November 4th, 1914.

Synopsis.

This paper deals particularly with the municipal water supplies, present and future, of the cities around the Bay of San Francisco, and incidentally of the so-called Hetch Hetchy project, for which Congress recently granted the necessary permits.

First, a detailed study is made of the population growth of the San Francisco-Oakland Metropolitan District and its several subdivisions. The conclusions reached are that, probably:

The East Bay portions will pass San Francisco proper in population, and to a greater extent in water requirements, in about 20 years;

The next census will show the entire district second to Greater Los Angeles; and that,

After a normal growth for two or three decades, Greater San Francisco-Oakland, and especially the East Bay district, will, for a few decades, grow very rapidly.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Then follows a brief description of existing systems and statements of their possible full development.

The question as to whether the Bay cities need and can afford a distant supply in the immediate future is then discussed.

Four or five decades hence, with the utmost co-operation of all interests, local supplies must be supplemented by water brought in from a distance.

The last two parts deal, respectively, with the considerations governing a wise choice of such distant supply or supplies, and the various possibilities.

The conclusions are that the problem is much more, and almost exclusively, one for the East Bay cities, than for San Francisco, and the desirability of considering the situation from this standpoint very carefully, before definitely fixing on the Hetch Hetchy project, is advised.

INTRODUCTION.

Urban communities on San Francisco Bay in California have proposed to secure additional water supplies. After more than a decade of endeavor, the City of San Francisco, acting in its own name, but latterly in behalf of a metropolitan water district, has just secured permission to use the Hetch Hetchy Valley and dam site on the main Tuolumne River, within the boundaries of the Yosemite National Park. The privilege of doing this was bitterly contested by numerous individuals, societies, and associations. Widespread interest was aroused, and much feeling engendered.

Because of the interesting nature of the problem to the Profession; because of the many engineering facts which have been brought out from time to time in a multitude of reports and hearings before Government officials and Congresses; and because various engineers have differed so widely in regard to many matters, a comprehensive paper on the subject is believed to be timely.

Topography of San Francisco Metropolitan Area.—San Francisco Bay, with its outlet to the ocean—the Golden Gate—is about in the middle of the California coast line.

The relief map, Fig. 1, shows the immediate region. The Northern Peninsula, generally referred to as Marin County, is quite hilly and

RELIEF MAP
OF THE
SAN FRANCISCO-
OAKLAND
DISTRICT
SHOWING PRESENT LOCAL
WATER SUPPLY SOURCES
AND
POSSIBLE EXTENSIONS
No. NAME
1 RICHMOND MUNICIPAL WATER DIST.
2 MARIN " "
GAUGING POINTS
3 SUNOL DAM 5 BRIGHTSIDE WEIR
4 NILES 6 BELMONT PUMPING STA.
EXISTING RESERVOIRS
CAPACITY, ACRE-FT.
7 CRYSTAL SPRINGS 59,000
8 PILARCITOS 3,306
9 LAKE MERCED 7,670
10 LAKE CHABOT 14,800
11 SAN ANDREAS 18,980
12 TEMESCAL 440
13 LAKE LAGUNITAS 740
14 PHOENIX GULCH LAKE 850
PROPOSED RESERVOIRS.
15 CALAVERAS 162,700
16 ARROYO VALLE 39,900
17 SAN ANTONIO 55,300
18 UPPER SAN LEANDRO 51,270
19 SAN PABLO 18,420
20 PINOLE 7,060
21 ALPINE 19,600
22 WILDCAT
ELEV., FEET
268
69
30
235
44
436
760
189
750
800
466
470
325
280
685

FIG. 1.

RELIEF MAP
SAN FRANCISCO
OAKLAND
AND VICINITY

mountainous, with a relatively small area suitable for urban development. The Southern Peninsula, on which is the City of San Francisco, has a much more favorable, but still unfortunate, topography for city building. The portion south of the corporate line, and as far as the City of Santa Clara, is locally known as the "Peninsula," only the eastern edge of which is level or relatively so. The mainland along the east shore of the Bay from Richmond south has a comparatively narrow strip of level area, changing to rolling, and then to sharply hilly, country. This increases in width toward the south. The total area suitable for urban development on this side of the Bay is many times greater than that in San Francisco and both peninsulas combined.

The range of mountains running along the California coast line—the Coast Range—really ends in Twin Peaks, within the corporate limits of San Francisco, and begins again just across the Golden Gate, extending thence northward.

East of the Bay, another mountain range, parallel to the Coast Range but higher and wider, extends across the entire region, with a very narrow break, known as Carquinez Straits, through which the combined waters of the Sacramento and San Joaquin Rivers are discharged into San Francisco Bay. In many ways this Mt. Diablo Range is a most important topographical feature of the San Francisco Metropolitan District. It modifies in a striking way the local climatic conditions; stands as a barrier against urban growth eastward; affords productive water-sheds and storage facilities for local water developments; and is a very serious obstacle to be overcome in bringing a water supply to the Bay region from those sections of the Sierra Nevada Mountains directly to the east.

Urban Development.—The Mission of San Francisco was established by the Franciscan Fathers on October 9th, 1776, practically in the geographical center of the present City of San Francisco. In 1846 there was only a little settlement, called Yerba Buena, on the shores of Yerba Buena Cove, between Telegraph Hill and Market Street. The name was changed to San Francisco in 1847. With the discovery of gold in California in 1848, the settlement suddenly became important. For a few decades the growth was phenomenal and then dropped to about the normal in cities of its size.

It is unfortunate that the original settlement was not on the mainland, as the available area for satisfactory city development on the Southern Peninsula is relatively small, and is sharply restricted by the topography.

Effect of Topography.—The effect of the local topography has been very marked. The disposition of the land masses affects the fog distribution and results in a peculiarly great variation in climatic conditions over the district. San Francisco's climate is foggy and windy, generally speaking, with summer temperatures only slightly higher than those of winter, the thermometer rarely reaching 80° Fahr., and even a few scattering snowflakes and thin films of ice are to be seen only at intervals of years.*

The city is compactly built, due to the limited area which is not on steep hillsides. As the climate is really quite equable, it is unfortunate that home building was not carried out along the same lines as in Los Angeles and all other California cities, namely, to accentuate and take advantage of the famous climate.

In 1874, the area was materially enlarged by the introduction of cable cars, which were developed to meet the local situation. A railroad built into the interior of the State in September, 1869, naturally ended on the east shore of the Bay, with ferry service into the city proper. The suburban service thus available to the mainland, started settlements in Alameda, Oakland, and Berkeley, where ample space, milder climatic conditions, and cheap individual water supplies resulted in a different type of city building.

At first ocean shipping was of pre-eminent importance, and the development was determined by deep-water wharfage. As railway transportation increased in relative importance, commerce was influenced by the railway terminals on the mainland, but the position of San Francisco as the commercial center has become so firmly established that it is doubtful whether it will ever be transferred across the Bay.

Effect of Geographical Location.—In the nature of the case, San Francisco's population is cosmopolitan and self-sufficient. The city very quickly became a metropolis, and was effectually isolated from any other center. It was the financial and commercial hub of western gold mining enterprise, which attracted the energetic and adven-

* The last snowfall was 1 in., on March 3d, 1896.

turous from all over the world. There are large foreign quarters in other cities of the country, but in San Francisco—except in Chinatown—the many nationalities are diffused. Consequently, its civic, religious, educational, and political ideas and institutions were and are different from those obtaining anywhere else in the United States. It is financially self-made, self-centered, and self-sufficient. The prevailing spirit was, and in large measure is yet, against San Francisco being a "cheap town." It is a wealthy one. Large personal freedom developed, and precedents of all kinds, including engineering methods, quickly came to be regarded lightly—often with excellent results.

In consequence, individualism was developed to an extreme, with a corresponding lack of co-operation. This condition obtains in a less, but still marked, degree to-day.

Considerable manufacturing necessarily developed, and, until the last decade, little was done on the Pacific Coast outside of the city. It became the center of Labor Unionism on the Coast, more particularly after the successful protests of Dennis Kearney and his followers against Asiatic labor competition.

Limits of Greater San Francisco-Oakland, as Used in This Paper.—The census adopts for metropolitan districts all urban population within 10 miles of the municipal boundaries. In this paper, however, the term will be used as including the territory which, logically, should ultimately have a common water supply. It is practically coincident with the region circumscribed by a circle of 30 miles radius with the San Francisco Ferry Building as the center. Greater Los Angeles, similarly, is considered as the area within a radius of 30 miles from the Los Angeles City Hall.

I.—Probable Future Population.

Growth of San Francisco.—For the past three decades the growth of the San Francisco Metropolitan District has been steady, and at the average rate for American cities of like population, but very much slower than Chicago, New York, and several others, when they were of similar size. The disastrous conflagration following the earthquake of April, 1906, temporarily affected San Francisco proper most seriously, but markedly accelerated the development of the East Bay cities and other suburbs. Such effect was not great enough to explain the relatively slow growth of Greater San Francisco-Oakland as com-

pared with all other important centers on the Pacific Coast and in the Far West during recent years.

The reasons for this, and how potent they will be in the future, are questions of primal importance. The population, and consequently the water requirements, are the first things to be determined in planning a water supply.

So far as the writer is aware, those who have estimated the future population of the district have not analyzed the peculiar conditions in California. The phenomenal percentages of increase in Los Angeles, Cal., Vancouver, B. C., Seattle, Wash., and other Pacific Coast cities in recent years, have never been taken into account, except by adding a statement that, remembering them, "the curve may be regarded as extremely conservative." This is not very illuminating.

Growth of California.—In the decade, 1900 to 1910, California's population increased 60.1 per cent. The rate for Oregon was 62.7%, Nevada, 93.4%, Idaho, 101.3%, and Washington, 120.4 per cent. The weighted average for these four other States was 95.9%, and for Washington and Oregon—the two other Pacific Coast States—95.8 per cent. The numerical increase was, Washington, 623 867; Oregon, 259 229; or together, 883 116—only 11 382 less than California.

Relative Growth of Northern and Southern California.—On examining detailed figures, it is found that the excessive population increase of the State is not of California generally, but of Southern California, consisting of the seven counties south of the Tehachapi—Ventura, Los Angeles, San Bernardino, Orange, Riverside, San Diego, and Imperial. The growth of population in these seven counties was 154%; in the rest of the State it was only 38%, and in the census San Francisco Metropolitan District but 41 per cent. The curves, Figs. 2 to 10, indicate that, unless something in the nature of widespread calamity or war occurs to change the respective rates of growth, Southern California will, by 1920, comprise more than half the State in all essential respects except area.

There are several reasons why this is surprising. They must in time be effective. These, in the probable order of importance, are:

1.—The natural resources of the State, like its precipitation, increase approximately in direct ratio from south to north. A marked

exception, whereby the reverse is true, is the attractiveness of the ocean beaches.

2.—The climatic conditions of Southern California, from a residence point of view, are inferior to those of the central and northern parts. This fact has been obscured by the effective advertising of Southern California. Greater San Francisco-Oakland is climatically

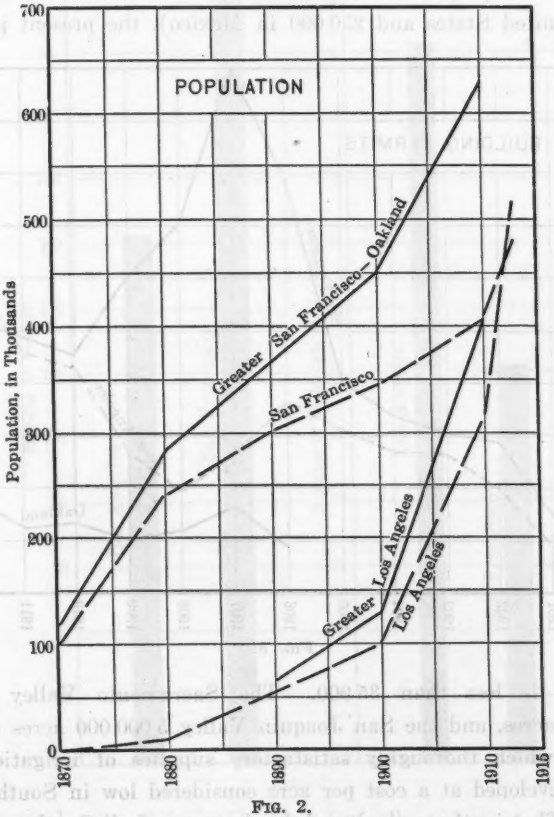


FIG. 2.

a markedly better summer resort than Los Angeles, and probably is not equalled anywhere in America.

3.—Within easy reach of San Francisco lies some of the most picturesque scenery in the world. There are opportunities to construct about the Bay cities the most attractive and beautiful roads

and boulevards on the Continent, and the Bay is almost unrivaled for yachting and motor boating the year round.

4.—The agricultural and horticultural possibilities of Southern California are literally dwarfed by those of the Central Valley—Sacramento and San Joaquin Valleys—alone. The total irrigated area of Southern California is a little more than 300 000 acres, exclusive of Imperial Valley (which will ultimately contain about 500 000 acres in the United States and 250 000 in Mexico), the present population

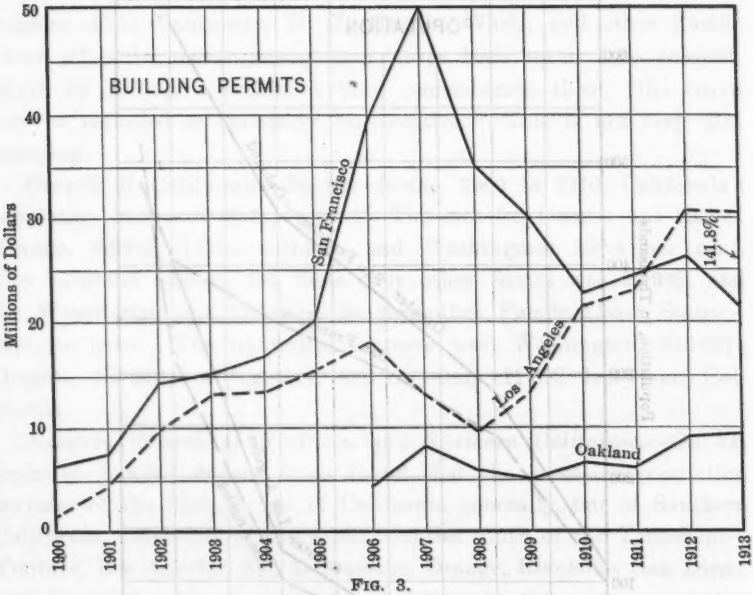


FIG. 3.

of which is less than 35 000. The Sacramento Valley contains 2 250 000 acres, and the San Joaquin Valley 5 000 000 acres of fertile land for which thoroughly satisfactory supplies of irrigation water may be developed at a cost per acre considered low in Southern California. That is, the agricultural development of all Southern California, which was the foundation for its entire development, is only 4% of the easily irrigable area of the Central Valley alone. Furthermore, the proportion of irrigable area adapted to growing oranges, lemons, grape-fruit, etc., is about the same in each region. When it is remembered that California produces 40% of the world's commerci-

ally grown oranges, and that the groves scattered about the Sacramento and San Joaquin Valleys have shown that oranges ripen there at least 6 weeks earlier than in Southern California, and suffer less from frosts, some realization of the situation can be obtained.

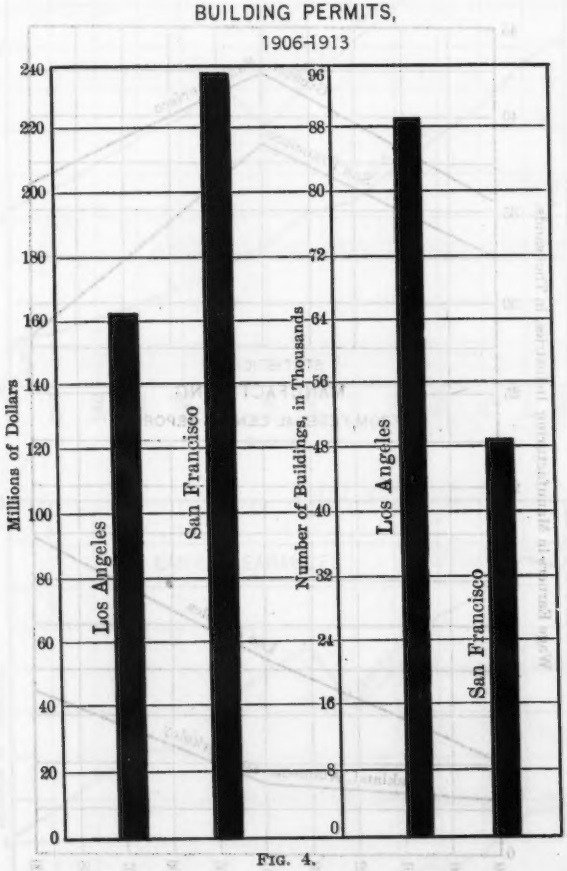


FIG. 4.

The total area of irrigable land logically tributary to San Francisco is 12 500 000 acres, and that of arable land is 15 393 000 acres.

5.—Thirty years ago, when Los Angeles had only 12 000 inhabitants—mostly Mexicans—San Francisco had 250 000, and was the "City" to which people "went down" from a vast region extending nearly 1 000 miles south, east, and north; it was practically the only

seaport on the Pacific Coast, and for more than 10 years was the western terminus of the only transcontinental railway.

In other words, the soil of Southern California is, to say the least, no more fertile; the water supply is by comparison pitifully

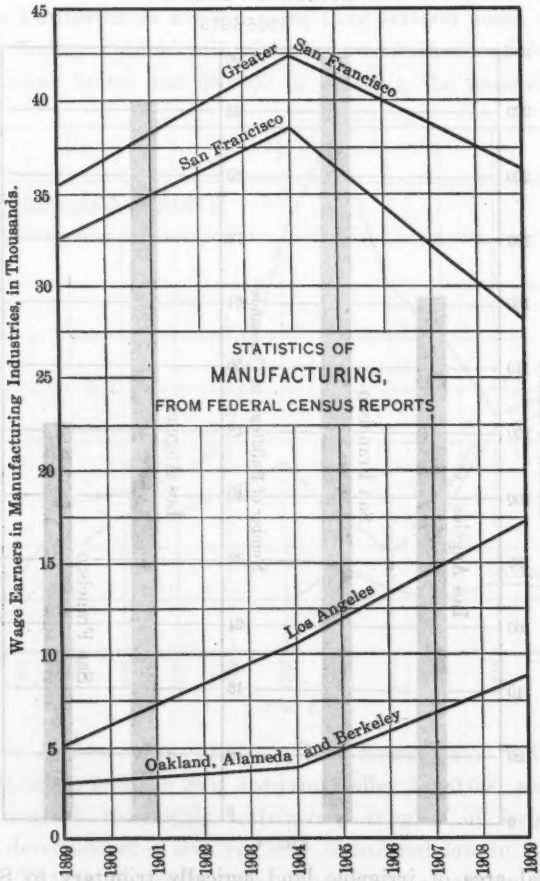


FIG. 5.

deficient; the climate not so delightful nor perfectly adapted to fruit culture, especially oranges and lemons; the extent of back country which can ever be made suitable for intensive farming, but a small fraction of that logically tributary to San Francisco; the natural

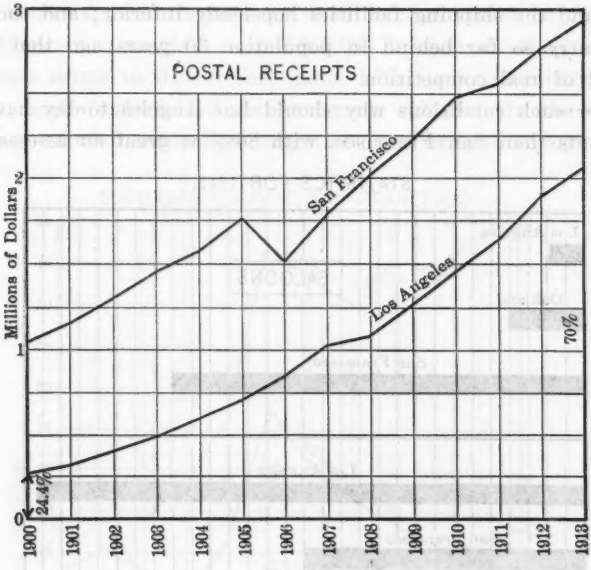

POSTAL RECEIPTS
Millions of Dollars
3
2
1
0
San Francisco
Los Angeles
70%
24.4%
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 6.

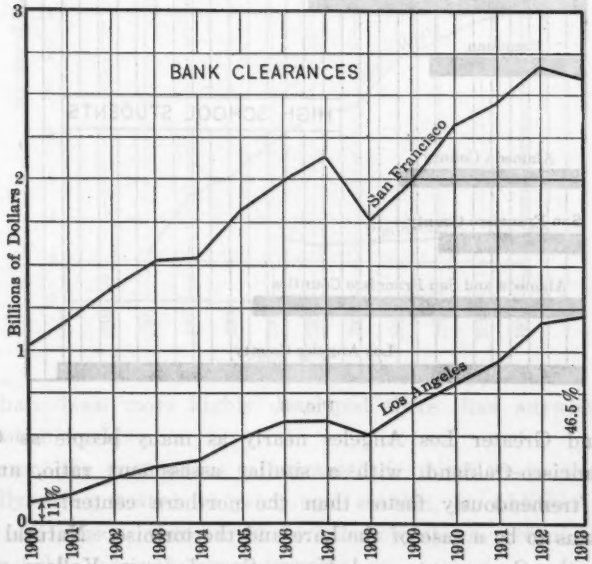

BANK CLEARANCES
Billions of Dollars
3
2
1
0
San Francisco
Los Angeles
46.5 %
11 %
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 7.

harbor and the shipping facilities hopelessly inferior; and the southern country so far behind in population 30 years ago that no one dreamed of real competition.

Under such conditions why should Los Angeles to-day have more inhabitants than San Francisco, with 86% as great an assessed valu-

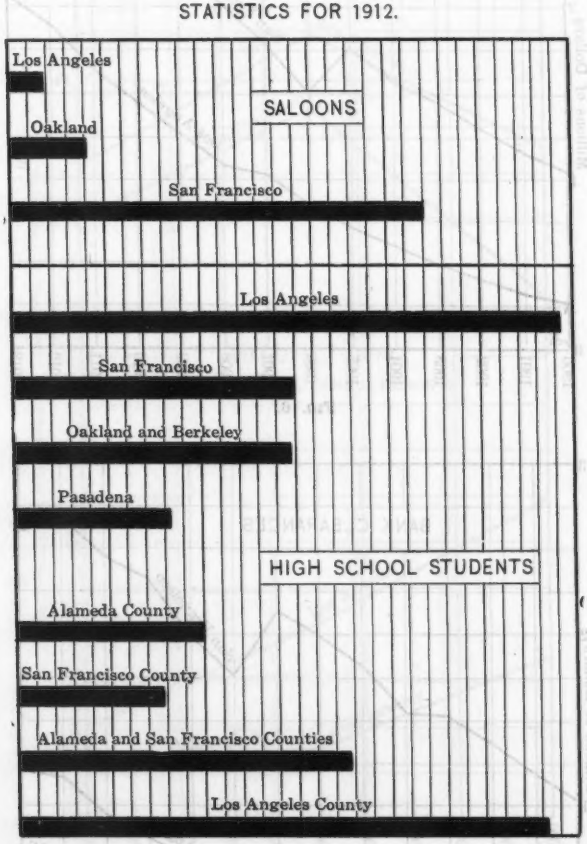


FIG. 8.

ation, and Greater Los Angeles nearly as many people as Greater San Francisco-Oakland, with a similar assessment ratio, and still growing tremendously faster than the northern center?

It seems to be a case of the hare and the tortoise. Natural advantages in the Sacramento and Upper San Joaquin Valleys resulted

in great bonanza grain farms—one man having raised on his own land holdings in a single season 1 000 000 bushels of wheat. It is an economic crime to devote such land, with its extraordinary crop adaptation and wonderful climate, to grain culture. In the Southland scanty rainfall compelled irrigation, and irrigation and citrus

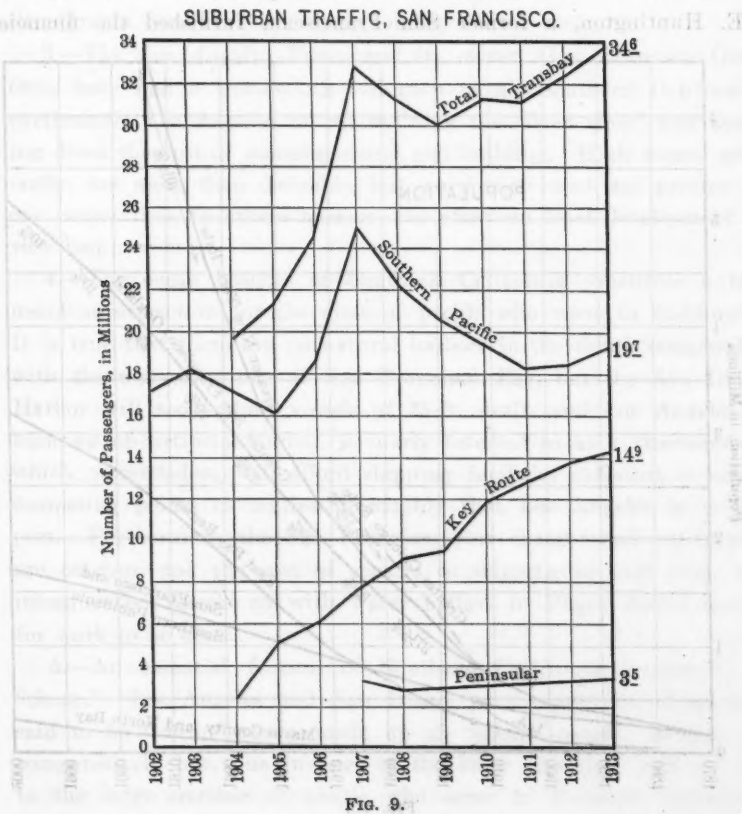


FIG. 9.

culture have been more highly developed there than anywhere else in the entire world.

However, the tremendous advantages of irrigation in the State were fully demonstrated 40 years ago, so that, in addition to this fundamental reason, there must be others quite as potent. These, in the probable order of importance, are:

1.—The Los Angeles Chamber of Commerce. This is doubtless the most efficient organization of its kind in the world, presenting a united, brave front in all matters affecting the development of, not only Los Angeles, but all Southern California. It is doubtless unequalled as an effective advertiser.

2.—The Pacific Electric Interurban System, for which Mr. H. E. Huntington, a former San Franciscan, furnished the financial

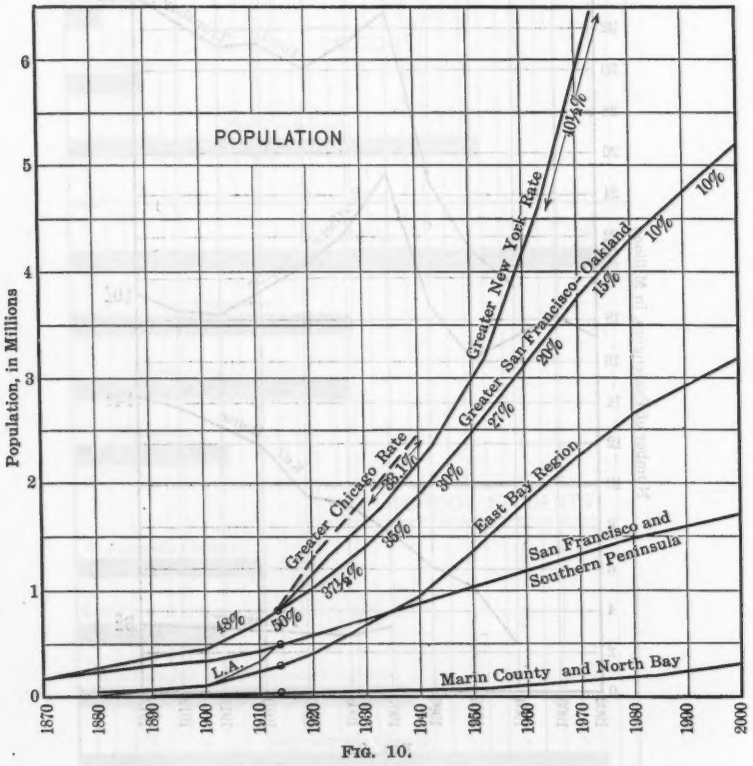


FIG. 10.

strength, and Mr. Epes Randolph, now of Tucson, Ariz., outlined the broad general plans. It is the most complete, extensive, well-constructed, and well-equipped system of interurban electric railroads in the world. There are 6 200 daily train movements over 47 routes, the total number of passengers carried yearly being more than 106 000 000. The total annual trans-bay suburban traffic between San

Francisco and Oakland, Berkeley, etc., is less than 35 000 000, or about one-third, in spite of the fact that such suburban service is, next to that of Los Angeles, the best and cheapest on the Continent.

In addition, there is doubtless no city in the United States—and that means the world—with a system of surface electric roads as modern and well-equipped as Los Angeles. The street railway situation in San Francisco is deplorable.

3.—The *Los Angeles Times* and its owner, Gen. Harrison Gray Otis, have had a tremendous influence in all Southern California, particularly Los Angeles, in maintaining the "open shop" and keeping down the cost of manufacturing and building. High wages, generally, are more than desirable, but when shop costs are greater in one center than in others near-by, the effect on local development is very bad.

4.—The ocean beaches of Southern California constitute a tremendous attraction for the class of people who come to California. It is true that there are no natural harbors in the South comparable with the marvelous one of San Francisco Bay, but the San Diego Harbor will soon admit vessels of 35-ft. draft, and Los Angeles is building an artificial harbor, jocularly referred to as a "harborette," which, nevertheless, will afford shipping facilities sufficient to cause competing points to realize thoroughly that Los Angeles is a seaport. Furthermore, the San Francisco port charges and regulations are onerous, and the cost of repairs to shipping so high that, not infrequently, vessels go with water ballast to Puget Sound points for work to be done.

5.—An essential feature of Southern California is that it is "cheap." Los Angeles and San Diego, particularly, are frequently said to be "cheap" towns made up of "cheap" people. Wages are comparatively low, due in part to the labor situation, and in part to the large number of people who come to Southern California determined to remain. All the larger places are surrounded by country topographically suited to indefinite urban expansion in all directions. Consequently, labor and rents are cheap. The expression "Southern Cafeteria," used derisively, has played a very important part.

6.—The fact that Southern Californians are often sneeringly referred to as "goo-goos" (goody-goody) has been an effective adver-

tisement. San Francisco, on the other hand, has been widely advertised as the "Paris of America." There are 200 saloons in Los Angeles and 2 200 in San Francisco proper, and Los Angeles has more than twice as many high school students.

7.—Until a decade ago, San Francisco was, politically, the State of California, and was practically governed by railway interests. Of all boss-ridden commonwealths in the United States, California took first rank, and bribery, "influence," and manipulation of public affairs for the Interests controlling the machine became accepted as a matter of course. The moral fiber of the community was seriously affected. The influx into San Francisco was relatively small, but the tremendous increase in the population of Los Angeles came from all over the United States, bringing a strong strain of Puritanism and higher civic standards. The result for Los Angeles is a community which can and does co-operate and trust its leaders, and secures vastly more advanced civic improvements, of which the water supply is the most striking.

The years of wrangling about San Francisco's water supply and private ownership of the present water system are not accidents, they are only some of the costs of its individualistic temperament.

8.—Because the newcomers quickly outnumbered the natives, Southern California is really not "Western" at all, but a sort of general average of the entire United States and Canada—particularly east of the Rocky Mountains—in education, political and religious ideals, institutions, and spirit. The large cost of railway transportation from New York to the Coast has kept the percentage of European immigration very low. The influx into the State naturally goes to that portion having the same civilization and ideals as the newcomers. Consequently, almost everybody in America has acquaintances, friends, or relatives in Southern California—often very many of them. The result is an "endless chain" never elsewhere equalled.

9.—This factor has been intensified by the limited areas available for intensive cultivation. No large irrigation or colonization schemes were there desirable or even possible. Small, entirely distinct and independent tracts were often settled almost wholly by people from one general community. For example, the Ontario Project, promoted in 1888 and carried to a successful conclusion by

Messrs. William and George Chaffey, formerly of Ontario, Canada, was largely settled by Canadians from that Province, and hence the name. The various settlements—Riverside, Redlands, San Bernardino, Ontario, Pomona, etc.—have retained their individuality. All newcomers have been pouring into small tracts having a gross area of about 300 000 acres, so that the real population density of the inhabited portions of the region is fairly high. The communities have been well established since 1895, and with practically no pioneering work since 1900. So far as municipalities are concerned, however, the reverse is true, in that there are unlimited areas available for urban growth in every direction.

10.—Due to the progressive spirit, the high average and extremely well distributed wealth, and the fact that the country is a tourist playground, automobiling in Southern California is more general than in any part of the world. This quickly resulted in the construction throughout almost the entire region of a system of beautiful boulevards. These have gone far to eliminate what little "pioneering" work was left to be done there a few years ago.

11.—All the scenic attractions of the Southland have been made easy and comfortable of access and brought emphatically to the attention of every tourist. Although the San Francisco Bay region naturally offers much more in every way except ocean bathing, and might be made one of California's greatest scenic assets, this certainly has not yet been done. Thus, Southern California has come to be regarded as the nation's playground and residence section *de luxe*, where the tourist spends his weeks and months while his visit to San Francisco is ordinarily of but a few days *en route*. For this reason it is extremely probable that the Panama-Pacific Exposition in San Francisco in 1915 will really benefit Southern California much more than the Bay region, especially as another peculiarly unique and attractive, though less expensive, Exposition will be held simultaneously in San Diego.

12.—California is in an earthquake region in the sense that *tremblors* are not infrequent. The damage from the shocks of April 18th, 1906, in the East Bay cities was insignificant. In San Francisco proper the chief damage was the breaking of the gas and water pipes in the filled-in areas, thus interrupting the main water supply. This caused one of the disastrous fires of history.

Property worth $350 000 000 was wiped out of existence, many of the inhabitants were widely scattered, and the normal development of the city was violently interrupted. Los Angeles, and to a much less degree, other Coast cities, gained thereby. The final effect on the size of the two cities, however, has been much less than is generally supposed. San Franciscans have nearly all returned. The financial status of many individuals was greatly altered, and the city as a whole is less wealthy than it would have been. Otherwise it, and particularly Greater San Francisco-Oakland, was benefited in many important ways. The effect of the disaster on the location of newcomers to the State was very marked for a time, and doubtless is still appreciable. However, even a great disaster rapidly fades from the world's memory. Los Angeles, quite as much as San Francisco, is in the *tremblor* district, and has fully as many tremors, and by the law of averages the northern city is the safer. Nowhere in the State has a really severe earthquake occurred in historic times; disastrous shocks are like lightning, in that they may strike anywhere. These facts are now generally understood and realized.

Class of Domestic Immigrants.—The people who come to California have not the slightest intention of pioneering, but rather of obtaining the pleasantest home within their means. This is a fundamental fact. That agricultural and suburban land values are from two to five times higher in Southern California than in the Northern sections has not yet had any marked effect, although, of course, it must in time.

It is of the greatest importance to realize that, 20 years ago, when the growth of Southern California began to be really phenomenal—not so much in percentage of population increase as in actual amount and character—the region was agriculturally and horticulturally quite fully developed. There were thrifty, well-matured groves and orchards covering more than two-thirds of the area now devoted to horticulture, with practically all the present steam railway mileage in operation, and beach, park, driveway, and scenic resort development done. In short, all "pioneering" was a thing of the past.

It was then that the future began to be discounted, boosting became general, the Pacific Electric System was built, the oil fields were exploited, and extraordinary growth commenced.

Probable Future Growth Within the State and of Greater San Francisco-Oakland.—To what extent and how quickly may these influences and conditions be expected to change, and with what results?

San Francisco is a big city, and its characteristics will change but slowly. Nevertheless, recently there have been some important forces in addition to those generally in operation. From within there are such people as the Rev. Dr. Aked, Bishops Hughes and Nichols, Jack London, Charles Tenney Jackson, Miriam Michelson, and a number of others, and the great University of California, which has the largest body of undergraduate students in America and strong graduate departments. Those parts of Greater San Francisco-Oakland having less of the Argonaut characteristics are rapidly gaining, and within a couple of decades will outnumber and constantly more deeply influence the main city. From without, the majority vote which the remainder of the State brings to San Francisco has forced woman suffrage, suppression of race-track gambling, throwing off railway political domination, etc. A red-light abatement act would have become a State law a few months ago had not a referendum petition held it up until the November, 1914, election, and San Francisco has already abolished its notorious Barbary Coast. By the initiative, next fall, California will vote on State-wide prohibition.

Thus Greater San Francisco-Oakland is quite rapidly reducing the advantages which Los Angeles urges so strongly, of a "wholesome atmosphere" for family life, although probably two or three decades will be required to secure any degree of parity in this regard between the two great centers.

It will doubtless be a decade before the Chambers of Commerce and Development Boards in Greater San Francisco-Oakland and Central California will speed up to the present Los Angeles standard. It will be at least as long before the present unsatisfactorily financed beginnings of an electric interurban system develop to the Pacific Electric standard, and longer before the natural attractions of the Bay region are fully utilized. The cost of living will probably always be lower in the Southland, and the beach advantages will never be offset, although the counter attractions of the northern Sierra Nevada Mountains will be made more and more important.

With rapidly increasing Far Eastern trade, the great San Fran-

cisco Bay must become an increasingly important factor. The route to the Orient from Puget Sound is more foggy and less safe, and from San Diego and Los Angeles, longer. San Francisco is the logical commercial center—the New York—of the Pacific Coast.

The Panama Canal will have an important effect, but for quite a few years much less than many seem to think. For a number of years other Coast cities will probably benefit almost as much as, if not more than, the Bay cities, particularly with lower manufacturing, handling, and ship repairing costs, due to lower wages and rents.

Within the last 4 or 5 years enormous areas in the great Central Valley have been thoroughly developed, from a land-and-water-company point of view, for sale in small tracts to home builders. The grand total is much more than 1 000 000 acres, equal or superior to, in every regard—except environment and convenience—and held at from one-half to one-fourth the prices of, lands with similar crop adaptation in Southern California. Unfortunately, the general idea has not been to use the land, but to build irrigation works and "colonize." Conditions have changed in the two or three decades since the Southland was developed. In 1880, 70% of the people in the United States were on the soil, to-day but 51 per cent. Subduing and farming irrigated lands is by no means a poor man's business, and in general those who have enough capital to succeed will not pioneer.

Already the large companies see the "handwriting on the wall" and are arranging to prepare, plant, and cultivate the land—develop it completely, and then sell farms and orchards, rather than raw land and agricultural and horticultural problems to individuals. A noteworthy instance is the Mills Orchard Company, which is planting about 10 000 acres of orchards, mostly lemons, at about the midpoint on the west side of the Sacramento Valley.

The State is putting $18 000 000 into a network of permanent highways, and the good roads movement is in full swing in the Central Valley.

Several careful students of conditions in California believe that the State is likely to have a single-tax law within the next decade, especially as the initiative is available. In this event the effect on the lands in question would be tremendous.

Taking all these things into consideration, it would seem that,

in about a decade, the great Central Valley will reach the point where Southern California was 20 years ago, and will develop thereafter in much the same way as the Southland is doing. Also, on account of the increasing tendency to concentrate in large cities, the effect on Greater San Francisco-Oakland will be much the same as the development of the southern country has been on Los Angeles. This in spite of the great extent of back country and the consequent development of relatively large local centers such as Sacramento.

Manufacturing on the Pacific Coast must also increase, but whether faster or slower than the agricultural and horticultural interests depends on several factors. Northern and Central California, Washington, and Oregon, particularly, can and will have more and cheaper hydro-electric power than any other place in the United States, and the rates and service will be, and already are, completely controlled and regulated by utility commissions. California has enormous quantities of fuel oil, but its cost is now equivalent to coal at about $4 per ton, and there is little probability of material reduction. The climatic conditions of San Francisco conduce to the highest labor efficiency on the Continent. The local market is important and rapidly increasing. The Panama Canal will make cheaply accessible enormously greater territory. San Francisco is the nearest point in the United States to the potentially tremendous consuming countries in the Far East.

The present high wage rates and unsatisfactory local labor situation make it unwise to expect very great increase in the near future. Nevertheless, it seems probable that the wage "differential" in favor of the other Far West cities will gradually be eliminated, especially in view of the general tendency toward State and National arbitration of labor disputes, laborers' compensation laws, etc.

It is impossible, of course, to evaluate numerically these many factors, but such is always the case in projecting population curves. Regarding Greater San Francisco-Oakland, it would appear that:

1.—There is no probability of excessive rates of growth for the next 10 or 15 years.

2.—The increase for 1910-1920 will be considerably greater than that for 1900-1910, during which the great fire occurred.

3.—The increase after about 1925 will be very rapid for several decades.

Probable Growth of Various Parts of Greater San Francisco-Oakland.—Of almost equal importance for the matter at hand is the division of the total growth between the main, east-bay, and suburban centers.

The influences resulting in the various local growths have been set forth, chief of which is available area. San Francisco has a population density of more than 10 000 per gross sq. mile, and probably 15 000 per net sq. mile. In Marin County, the area available for city development within a reasonable time zone is relatively small and scattered, so that the suburban traffic facilities are, and, for some time, will probably continue to be, below the other local standards. Nevertheless, the climatic conditions are peculiarly attractive and the region so picturesque as to assure great suburban development of a high grade, with excellent transportation service. The population down the Peninsula will be much denser, but always essentially suburban to San Francisco alone. At the south end of the Bay is San José, as old as San Francisco, about 45 miles from it and from Oakland. It is now, and doubtless for several decades will be, a local center rather than a suburb.

The trans-bay cities, Oakland, Alameda, Berkeley, etc., which are physically one city, were the result of San Francisco's natural barriers, in spite of a suburban trip so onerous as to begin to alienate them when well grown. Another vital factor is the mainland transcontinental, and the most important of the local railway terminals and shops. Fig. 9 shows the trans-bay traffic. After the disaster in April, 1906, very large numbers of San Franciscans went to suburban points for temporary shelter, and gradually came back home in 1908. Since then this factor has been unimportant.

Although the East Bay cities are growing at rates somewhat like that of Southern California, and very much faster than the main city, they are also developing a greater independence and self-sufficiency. The social and political conditions existing in these communities are essentially different from those in the main city.

A bridge from San Francisco to the mainland has often been suggested, in fact a bill is now before Congress authorizing one. The trans-bay traffic indicates that such a structure, or a "tube"—which would be much more expensive—is not economically justified, and probably will not be for a long time.

The curves, Fig. 10, show the writer's conclusions from consideration of a great number of elements, chief of which, however, are those just given. All urban population estimates for a period of more than one-third of a century have a sharply limited value. Revolutionary changes, due to inventions, different ideals and tastes—such as a real "back-to-the-soil" movement would typify—cannot be anticipated.

II.—Probable Water Consumption.

Average Daily Consumption Per Capita.—The per capita consumption of water seems to be constantly increasing in all great cities of the United States, and at a somewhat slower rate in most European cities, largely due to higher standards of living. When meters are installed, large water wastes stop, causing an immediate drop in consumption, but an increase at once begins. There is no reason why liberal use should not be encouraged. This necessity and luxury of modern life costs less than any other, when compared with the real comfort it affords. At any rate, the increased consumption in American cities has greatly exceeded the expectations of 20 years ago.

San Francisco has a relatively low per capita consumption, which, in spite of the cool summers, warm winters, and foggy climate, can hardly continue. Wages are high, and people spend freely for comforts. Outside San Francisco proper there will be a marked increase in the use of water for lawns and gardens.

John R. Freeman,* M. Am. Soc. C. E., concludes that the use in Greater San Francisco 50 years hence will be "125 to 150 gallons per capita per day, although by that time substantially every service is metered." The probable curves for all component parts and for the entire Metropolitan District are given in Fig. 11. If anything, the quantity for San Francisco proper is higher in comparison than for the other parts of the district.

Probable Gross Requirements of Greater San Francisco-Oakland and Its Component Areas.—Ordinarily, combining the population and daily per capita consumption curves gives the gross water requirement, but, in semi-humid and arid territories, this is not quite the case. In this district a considerable area is devoted to agricultural purposes, not a little of which will be used in this way,

* "The Hetch Hetchy Water Supply for San Francisco, 1912;" Report by John R. Freeman, San Francisco, July 15th, 1912, p. 79.

even when the population of the district reaches 5 000 000. Until required for city uses, these lands will be cultivated more and more intensively with consequently greater reliance on artificial and controlled watering.

The underground resources made available by wells and the local surface run-off naturally tributary to the region and which can be conserved at a reasonable cost, would be considerably more than sufficient for the arable land. However, much of such water is used in the main and East Bay cities, and it is impossible in considering the water situation of the district to separate the two kinds

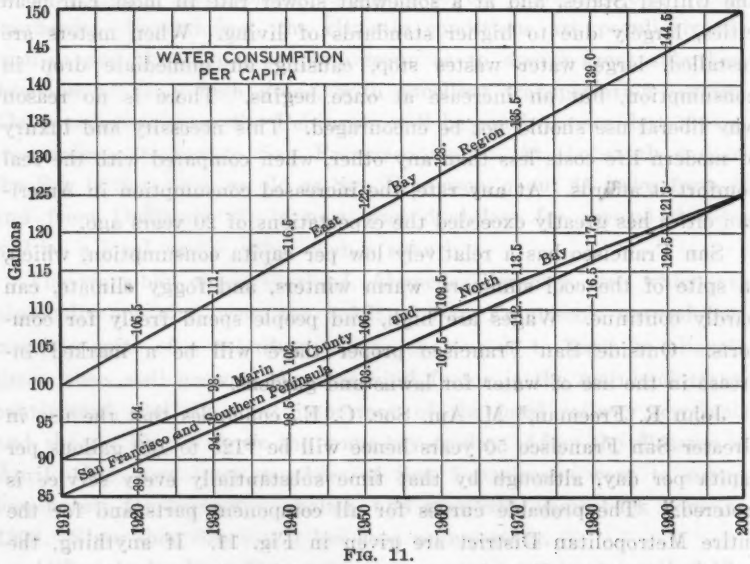


FIG. 11.

of requirements. Consequently, the easy and apparently direct way is ordinarily to obtain the gross water requirement by combining the urban and irrigation uses. Subtracting from these the quantity which may be secured locally, gives the quantity to be brought from a distance.

In doing this, however, it must be borne in mind that well water now available for irrigation and quite satisfactory in quality for domestic purposes will become unsatisfactory for the latter use without effective purification, when the region becomes urban in char-

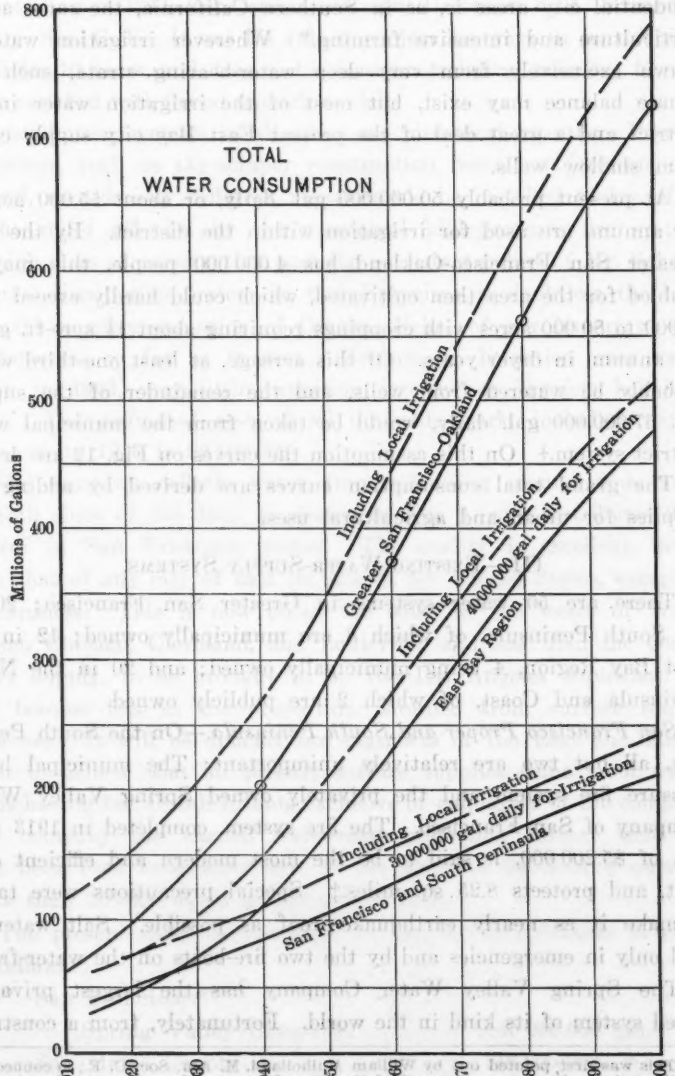

TOTAL
WATER CONSUMPTION
Millions of Gallons
800
700
600
500
400
300
200
100
0
1910
1920
1930
1940
1950
1960
1970
1980
1990
2000
Including Local Irrigation
Greater San Francisco-Oakland
Including Local Irrigation
40 000 000 gal. daily for Irrigation
East Bay Region
Including Local Irrigation
30 000 000 gal. daily for Irrigation
San Francisco and South Peninsula

FIG. 12.

acter. Hence, it is not correct to assume that the duty of water in residential city areas is, as in Southern California, the same as for horticulture and intensive farming.* Wherever irrigation water is drawn exclusively from very deep water-bearing strata, such fortunate balance may exist, but most of the irrigation water in the district and a great deal of the present East Bay city supply comes from shallow wells.

At present probably 50 000 000 gal. daily, or about 55 000 acre-ft. per annum, are used for irrigation within the district. By the time Greater San Francisco-Oakland has 4 000 000 people, this may be doubled for the area then cultivated, which could hardly exceed from 75 000 to 80 000 acres with croppings requiring about 1½ acre-ft. gross, per annum, in dryer years. Of this acreage, at least one-third would probably be watered from wells, and the remainder of the supply, say, 67 000 000 gal. daily, would be taken from the municipal water district system.† On this assumption the curves on Fig. 12 are drawn.

The grand total consumption curves are derived by adding the supplies for urban and agricultural uses.

III.—Existing Water-Supply Systems.

There are 50 water systems in Greater San Francisco; 20 in the South Peninsula, of which 8 are municipally owned; 12 in the East Bay Region, 4 being municipally owned; and 20 in the North Peninsula and Coast, of which 2 are publicly owned.

San Francisco Proper and South Peninsula.—On the South Peninsula, all but two are relatively unimportant: The municipal high-pressure fire system and the privately owned Spring Valley Water Company of San Francisco. The fire system, completed in 1913 at a cost of $5 200 000, is said to be the most modern and efficient ever built, and protects 8.25 sq. miles.‡ Special precautions were taken to make it as nearly earthquake-proof as possible. Salt water is used only in emergencies and by the two fire-boats on the water-front.

The Spring Valley Water Company has the largest privately owned system of its kind in the world. Fortunately, from a construc-

* This was first pointed out by William Mulholland, M. Am. Soc. C. E., in connection with the estimated water requirement of the City of Los Angeles, when the Los Angeles Aqueduct was under consideration. Practically none of the territory marked out for the future development of Los Angeles secured irrigation water from shallow wells.

† C. D. Marx, M. Am. Soc. C. E., adopts 95 000 000 gal. daily for a much greater district, including a considerable part of Santa Clara County.

‡ The City and County of San Francisco have a total land area of 47 sq. miles.

tion point of view, all planning and building for a period of 43 years ending in 1911, were under one man, Hermann F. A. Schussler, M. Am. Soc. C. E. It was his rule to project his work in considerable detail for 10 years, and, true to his German technical training, he was never satisfied with "good enough work."

Distribution systems in all cities are piecemeal growths. Present conditions, such as sky-scraper construction, were not foreseen. In general, considerably different plants would be installed to-day. Due to the relief afforded by the municipal high-pressure fire system, the high class of construction, and the consistent policy of Mr. Schussler, the distribution system in San Francisco is doubtless the best, with the possible exception of Omaha, Nebr., of any important city in America, so far as it goes. Because of litigation over rates, begun in 1903, very little extension into newer districts has been made. The estimated cost of all desirable work, including extensions, up to 1920 is $2 500 000.

The company collects water from adjoining mountainous areas on both sides of the Bay, as shown in Fig. 1, and distributes 95% thereof in San Francisco proper. The quality is excellent, better than that of any city of half its size in the United States, except as to hardness. This is now about the same as the water of Washington, Chicago, Cleveland, and Louisville, and less than the Owens River supply, to be brought in by the Los Angeles Aqueduct. It will become less as the proportion obtained from surface sources increases. It will be difficult for engineers of the East and Middle West to believe that all present surface supplies come from watersheds having but 18 inhabitants exclusive of employees of the company. The company has carried water-shed protection farther than has ever been even considered east of the Rockies, purchasing outright more than 100 000 acres of water-shed land.

The present daily consumption on the South Peninsula is about as follows:

San Francisco:*	
Spring Valley Company	40 000 000 gal.
San Mateo County	16 000 000 "
Part of Santa Clara County	10 000 000 "
	66 000 000 gal.

* In addition wells and Bay supplies are used in manufacturing, to an unknown extent.

East Bay Region.—On the mainland, the important factor is the Peoples Water Company, which supplies practically all the water used in Oakland, Alameda, Berkeley, and several smaller places. Unlike the Spring Valley Water Company, it is the successor of nine lesser companies which entered the field during the past 45 years. These companies, becoming unable to meet the rapidly increasing demand, were successively displaced by stronger ones. According to J. H. Dockweiler, M. Am. Soc. C. E., who has been Consulting Engineer for the Cities of Oakland and Berkeley for a number of years, the history of these attempts to keep pace with the widely scattered and rapidly growing population centers by private capital is a most interesting and curious succession of failures properly to grasp opportunities and protect investments by adequate preparation for the future. According to him, largely as a result of 5 years' relentless and wasteful competition between two such companies, the distribution properties are in a considerable degree superimposed on each other; short-lived materials have been used extensively, and the mains and reservoirs are out of consistent relationship with the topography of the territory and the developed sources of supply. According to his records, 48% of the nearly 900 miles of pipe are 2-in. and less, and only about 30% are of cast iron. The area served is 67 sq. miles, 27 of which are above the 200-ft. contour, and it is divided into sixteen pressure zones or areas, ranging from sea level to 1 000 ft. elevation. The system has 60 000 connections, 95% of which are metered, relatively few fire hydrants, and average receipts of 25 cents per 1 000 gal., with base rates as high as 35 cents.

About 100 blocks of the business section of Oakland are covered by a municipal fire system costing, to date, $136 000. Elsewhere on the mainland the fire protection is, in general, very poor, and in many areas negligible. The situation in this regard is probably as unfortunate as in any important center in America.

About one-third the water comes from the surface supplies of the company, shown in Fig. 1, and almost all the remainder from wells near the Bay shore. According to Mr. Dockweiler, the quality is generally satisfactory, although hardly up to the standard of that supplied to San Francisco. The present daily consumption is:

Peoples Water Company:	
Surface	7 000 000 gal.
Underground	11 000 000 "
Union Water Company	700 000 "
Other systems	3 000 000 "
Private wells, about 4 000	8 000 000 "
	29 700 000 gal.

This is exclusive of irrigation water.

Marin County and North Bay Region.—The four principal systems are the Marin Water and Power Company, the North Coast Water Company, the Vallejo Municipal Water-Works, and the Benicia Water Company, the latter being privately owned. The first two are supplying about 2 000 000 gal. daily from surface sources, and the others furnish water for Vallejo (population, 15 000) and Benicia (population, 3 000).

IV.—Maximum Safe Yields of Existing Sources.

The maximum safe yields obtainable by the complete economical exploitation of the various local developed and undeveloped sources are about as follows:

San Francisco and Peninsula.—The complete development of the Spring Valley Water Company's sources of supply would require creating the Calaveras Reservoir, by building a dam 220 ft. high, work on which is now in progress; two other reservoirs, San Antonio and Arroyo Valle, all in the Alameda Creek water-shed; and the diversion of other Coast streams into the Peninsula reservoir system.

The Calaveras Reservoir, estimated cost, $1 500 000, will have a capacity of 53 000 000 000 gal., or 162 700 acre-ft. The flow-line elevation will be 790 ft., and the contributory water-shed is 100 sq. miles, or with the Upper Alameda water-shed, 140 sq. miles.

The San Antonio Reservoir, estimated cost, about $750 000, will have a capacity of 11 500 000 000 gal., or 35 300 acre-ft. The flow-line elevation will be 455 ft., and the contributory water-shed, 38.7 sq. miles.

The Arroyo Valle Reservoir, partly storage and partly regulating, is estimated to cost about $800 000, and to have a capacity of 13 000 000 000 gal., or 39 900 acre-ft. Its flow-line elevation will be 800 ft., and the

contributory water-shed, 140 sq. miles. It is estimated that the related development of underground sources in Pleasanton Valley and at Sunol will cost $1 300 000.

The Peninsula development will intercept the run-off from 65 sq. miles of territory, contributory to Pescadero and San Gregorio creeks, and convey it by canals, pumps, and tunnels into the Crystal Springs Reservoir, at an estimated cost of $8 500 000 for 50 000 000 gal. daily, or $170 000 per 1 000 000 gal. daily.

The maximum yield which could thus be economically secured for use in San Francisco proper has lately been the source of much bitter controversy. Since 1910 an extraordinary number of engineers, usually believed to be especially competent in such matters, have made examinations and have reported widely differing results, as indicated in Table 1.

There are complete records of gauge heights and draft on reservoirs as follows:

Crystal Springs .....1888 to date.
San Andreas .....1870 " "
Pilarcitos (Except totals only for 1889 and 1898-1902) .....1867 " "
Lake Merced (With several breaks prior to 1906) .....1883 " "

There are run-off and discharge records as follows:

Niles Dam .....1888 to 1900.
Sunol Dam .....1900 to date.
Sunol Aqueduct at Brightside Weir....1906 " "
Pumping records at Belmont Station (With some breaks prior to 1898)..1888 " "
Calaveras Creek at dam site.....1898 to 1908, and 1911 to date.

Fig. 13 shows the continued run-off data at the Crystal Springs, San Andreas, and Pilarcitos Reservoirs, platted on arithmetic-probability scales, according to the method proposed by Allen Hazen, M. Am. Soc. C. E.* The consistency of the data thus shown makes striking the wide variation of engineers' estimates in Table 1.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1539.

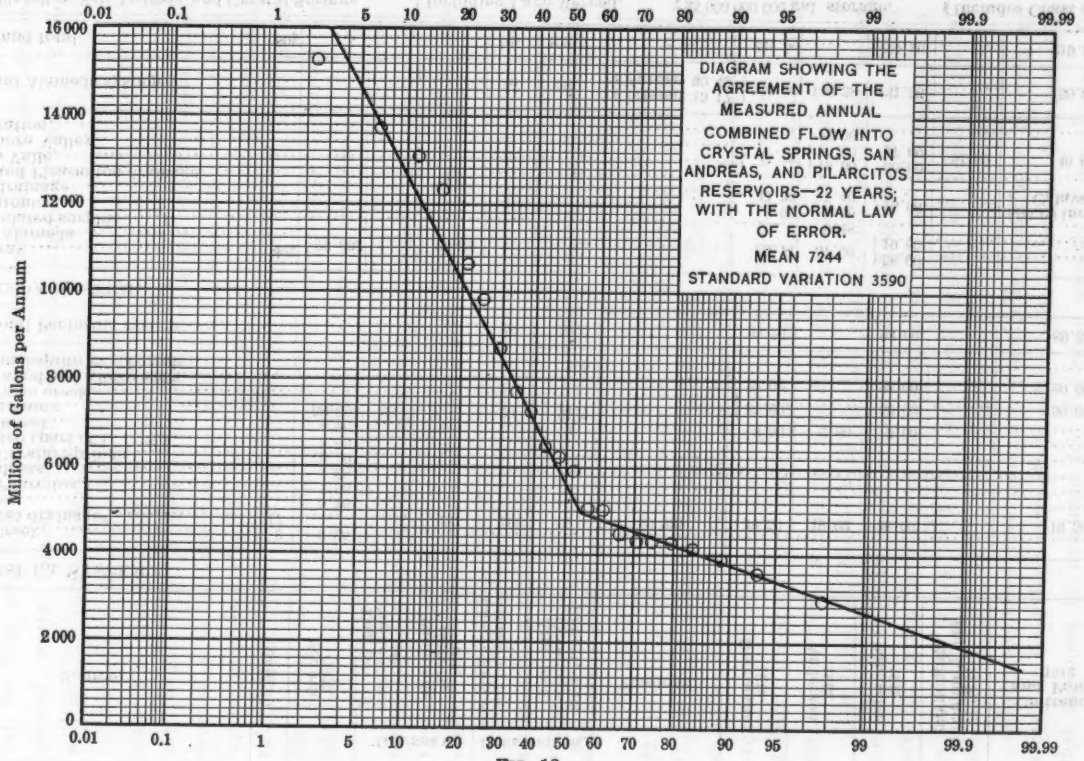


DIAGRAM SHOWING THE AGREEMENT OF THE MEASURED ANNUAL COMBINED FLOW INTO CRYSTAL SPRINGS, SAN ANDREAS, AND PILARCITOS RESERVOIRS—22 YEARS; WITH THE NORMAL LAW OF ERROR.
MEAN 7244
STANDARD VARIATION 3590
Millions of Gallons per Annum
16 000
14 000
12 000
10 000
8 000
6 000
4 000
2 000
0
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99

FIG. 13.

TABLE 1.—RESOURCES OF THE WATER SUPPLY OF GREATER SAN FRANCISCO-OAKLAND.

Source.	Mendell.	Scowden, 1875.	Grunsky. Eng. Soc. of Pacific Coast, 1908.	Grunsky. Tuolumne Report, 1902.	Dockweiler. Long's Report to Secy. of Int., 1910.	Dockweiler. American Cosumnes Report, 1912.	Freeman.	Herrmann, 1912.	Anderson, 1912.	Schussler, 1912.	Mulholland and Lippincott. 1912.	Chittenden and Powell, 1912.
PENINSULA SYSTEM.												
Lobos Creek	2	2.25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
San Mateo drainage*	17	.....	.....	17	21†	19	20-23†	19.50	19.02	19.00	.....	19.50
Pilarcitos	....	5.16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Lower Pilarcitos	....	1.93	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
San Andreas	....	4.28	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Upper Crystal Springs	....	15.00	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
San Mateo (part of L. C. S.)	....	3.23	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Lake Merced	....	4.267	.....	2	.....	3	.....	3.50	2.80	3.00	.....	.....
Coast streams	....	58.00	20	.....	20	31.7	.....	51.20	.....	50.00	.....	50.00
West Union creek	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Ravenswood and Aloiso wells	....	.....	.....	.....	.....	.....	.....	21.00	.....	21.00	.....	20.00
San Francisquito cr. (Portola)	....	.....	.....	7.5	7	5.8	.....	.....	.....	.....	.....	.....
Total Peninsula system	....	.....	.....	.....	48	59.5	.....	95.20	.....	93.00	.....	89.50
ALAMEDA SYSTEM.												
Calaveras	30‡	79.692	.....	25-32	.....	.....	.....	60.14	57.00	58.00	.....	.....
Upper Alameda	....	.....	.....	.....	.....	.....	.....	.....	.....	10.00	.....	.....
Accumulated surplus	....	.....	.....	.....	.....	.....	.....	.....	9.48	.....	.....	.....
San Antonio	....	.....	.....	3-5	.....	.....	.....	8.92	8.50	15.06	.....	90.00 includes Calaveras.
Sunol drainage	....	.....	.....	.....	.....	.....	.....	11.36	7.40	.....	.....	.....
Sunol and Pleasanton drainage	....	.....	.....	12	.....	.....	.....	.....	.....	.....	.....	.....
Arroyo Valle	....	.....	.....	.....	.....	.....	.....	45.38	18.00	30.00	51.50	40.00
Livermore Valley	....	.....	.....	.....	.....	.....	.....	.....	30.00	17.60	.....	.....
Evaporation	....	.....	.....	.....	.....	.....	.....	.....	12.00	.....	.....	.....
Total Alameda system	....	.....	.....	.....	57	50	Present 15.17 Future 30.40	125.80	142.38	131.26	.....	130.00
Grand total	80§	.....	.....	.....	105	109.5	.....	221.00	.....	224.26	.....	219.50

* Pilarcitos, San Andreas, and Crystal Springs. † Including Lake Merced. ‡ 25 000 000 000 gal. storage. § Includes Coast streams connected with enlarged (Lower) Crystal Springs Reservoir and also surplus from Calaveras, but does not include San Antonio, Arroyo Valle, Pleasanton, or Sunol gravels.

The territory from Greater San Francisco's southern limits on both sides of the Bay to the south end of the Santa Clara Valley can be fully supplied from the remaining local surface and underground water sources within reasonable limits of cost. There will be little excess, however.

East Bay Region.—The maximum safe yield of underground sources is problematical; it depends primarily on the extent to which the various fruitful underlying water strata are supplied from local sources, such as delta fans and cones, and from distant exposed Pleistocene gravels. Considerable difference of opinion exists regarding this matter, also. Definite conclusions are not yet justifiable as to the extent to which the present draft of probably 12 000 000 or 13 000 000 gal. daily may be safely maintained or increased.

The Peoples Water Company proposes to increase the local surface water supplies by building three dams on San Pablo, Upper San Leandro, and Pinole Creeks, thus creating three collecting and storage reservoirs.

The first of these it is estimated will cost about $2 335 000, have a capacity of 6 000 000 000 gal., or 18 420 acre-ft., and a flow-line elevation of 325 ft. The water-shed is 34.65 sq. miles and the estimated safe yield 8 000 000 gal. per day.

The second reservoir, on San Leandro Creek, estimated cost $2 878 000, would have a capacity of 16 700 000 000 gal., or 51 270 acre-ft., and a flow line at an elevation of 470 ft. The water-shed is 32.94 sq. miles. Below it is the present Lake Chabot or San Leandro Reservoir from which more water is wasted than impounded in seasons of average precipitation. The upper reservoir would conserve all the run-off, and thus increase the safe daily draft about 10 000 000 gal.

The third, on Pinole Creek, the last which would be created, would cost about $676 000, have a capacity of 2 300 000 000 gal., or 7 060 acre-ft., and a flow line at an elevation of 290 ft. The water-shed is 10.5 sq. miles, and the safe draft would be about 3 000 000 gal. daily.

These proposed reservoirs are practically contiguous to built-up areas. In time their water-sheds will be built on, at least, it is desirable that the area be available for the rapid growth of population. It would be necessary to filter the water carefully. The utilization of the San Pablo and Pinole sites would affect the annual water replen-

ishment in the delta cones of these creeks where a considerable number of wells are in use. The quantity of water required to be delivered to such regions as recompense, would be about 1 500 000 gal. daily.

The Richmond Water District plans are for a distant supply from the Sacramento River.

Marin County and North Bay Region.—The plans outlined by the Marin Municipal Water District provide for a total safe draft from local sources of 14 000 000 gal. daily, which is regarded as the maximum feasible limit. In the North Bay region the available increase from local water sources is relatively insignificant.

Summary.—The total ultimate safe draft for municipal purposes, of a quality quite satisfactory and well above the present standards for important cities in America, and obtainable at a reasonable cost by complete development of local sources is, then, about as follows:

For San Francisco and Peninsula.....	180 to 200
" East Bay Region.....	37 to 40
" Marin County and North Bay Region....	14 to 16
	231 to 256

V.—CAN GREATER SAN FRANCISCO-OAKLAND AFFORD A SIERRA SUPPLY?

San Francisco proper has available a disproportionately large part of the local water yield. By complete development, which can be done piecemeal as necessary, it can have an ample supply of excellent quality for at least 75 years to come, exceptionally large local reservoir capacity, and consequently relatively small upkeep and danger of interruption from floods, landslides, or earthquakes.

The city's present assessed valuation is $623 000 000. To acquire the Water Company's properties needed will cost between $35 000 000 and $40 000 000. The city has not finished rebuilding following a $350 000 000 fire. It has a total authorized bond issue of $43 000 000, exclusive of the Hetch Hetchy issue of $45 000 000 which latter is authorized, but not put out. Because of an exceedingly bad transportation situation, it is well started on a programme of taking over and operating all traction lines within its limits. This will require an estimated expenditure of between $100 000 000 and $125 000 000 during the next 25 years. It has no elevated or subway systems, though these are needed and cannot long be postponed. It is a rapidly growing city, with

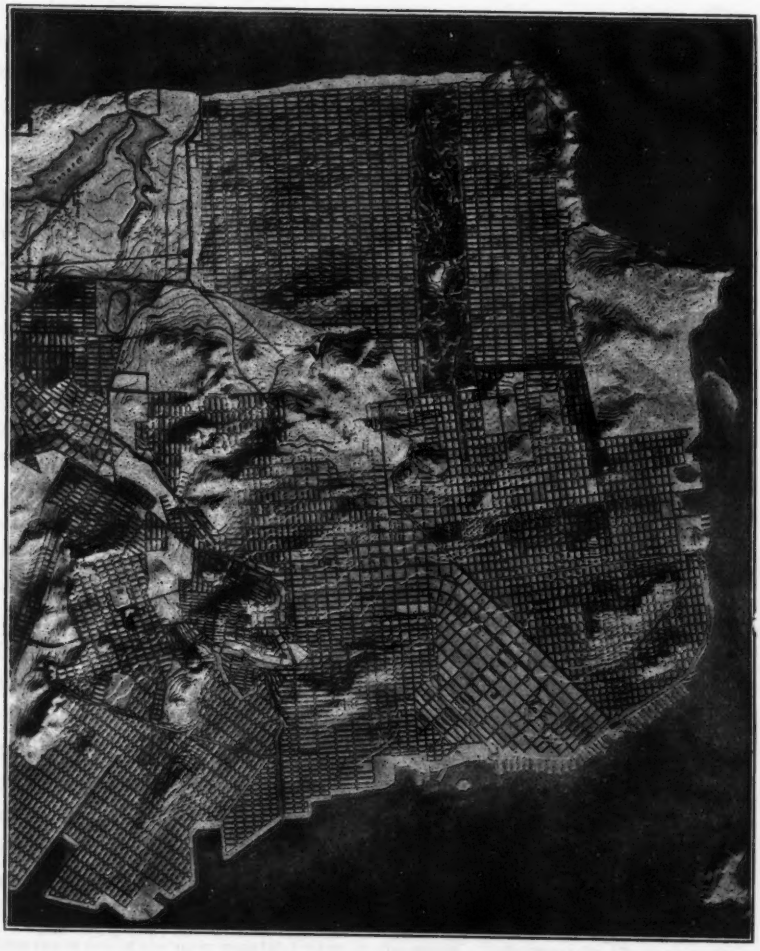


FIG. 14.—RELIEF MAP OF SAN FRANCISCO, SHOWING STREET LINES IN RELATION TO TOPOGRAPHY.

all the financial needs that this implies, yet is already surpassed in population by Los Angeles, which, 15 years ago, was not even admitted to be a rival. Its topography necessitates heavier costs for improvements than in almost any other city in America, and more to be done.* The local labor and political situation makes labor costs high. The temperament of its people makes concerted action difficult and progression slow and along indirect and expensive lines.

The outstanding indebtedness of Los Angeles is about 4% greater than that of San Francisco, with about seven-eighths the assessed valuation. However, Los Angeles owns its local water system, and has finished bringing in the Sierra supply. San Francisco, at least, will have to double its outstanding bond issue, in order to acquire the Spring Valley Water Company's properties, and the proposed completed Hetch Hetchy supply will add more than twice as much. At the best, then, to put the two centers on a proper basis of comparison, including the Sierra supply, San Francisco's bonded indebtedness would be more than four times that of Los Angeles. As to ability to care for such indebtedness in the future, Los Angeles is, in this regard, as in population, growing tremendously faster.

Under such circumstances, the real question is: Can San Francisco afford the luxury of a Sierra water supply at an immediate expense of more than 10% and a total ultimate cost of at least 16% of its present assessed valuation?

San Francisco's natural destiny as the commercial center of the Pacific Coast is beginning to be threatened. At least, until this matter is settled one way or another, San Francisco proper will be the largest and richest portion, and the natural leader of the Bay communities in this struggle.

In competition for newcomers, and in other elements causing a city's growth, is the water supply, as long as it is well above the American standard in quality and is delivered in ample quantities, at reasonable cost, and under satisfactory pressures, an aggressive and

* The topography and natural features of San Francisco are such that it could have been made one of the most beautiful cities in the world. Unfortunately, like many other places, it grew "Topsy fashion," and the layout is rectangular, quite regardless of the configuration of the land. There result grades as high as 22 per cent. The opportunity which the great fire of 1906 presented for re-designing the most important part of the city was not utilized. The City Engineer's office recently stated that $26 000 000 was the cost of certain improvements, necessary under the circumstances, but which would have been entirely avoided had the city's planning been adjusted to the topography, as might have been done, and that this expenditure was but the beginning of a series which from time to time must be made.

obvious factor, as compared with local transportation facilities, modern marine and railway terminals, paving, parks, boulevards, speedways, making areas spoiled by local development available for higher classes of city building, etc.?

Los Angeles, with 86% of the assessed valuation, as a matter of absolute necessity, has brought in a supply of 250 000 000 gal. per day from the Owens River at a cost of a little more than one-fourth of San Francisco's present programme. The quality of this water will, at best, be no better than the local supplies of San Francisco. Perhaps most important of all, however, is the fact that from the very outset, every drop of the water brought in will be used, and at prices well above the existing sinking fund and operation charges per million gallons daily. The reason that this is possible in Los Angeles, and utterly impossible as to Greater San Francisco-Oakland, is that irrigation is absolutely essential in the environs of the southern city, and irrigation water even at the city base rate of 9 cents per 1 000 gal., would be only $29 per acre-ft., which is well within the commercially feasible limit in that territory; on the other hand, in the environs of San Francisco-Oakland, little water is imperatively required, and none could be sold at even $5 per acre-ft. The fact that irrigation requirements for lands adjoining Los Angeles are practically the same as the urban needs will be when the city spreads out over them, has already been mentioned. If engineering is attaining the best results for the expenditures, this question is an engineering one of the highest type. It is the proper valuation, for a great city's purposes, of two sources of water supply.

East Bay Region.—The situation of the mainland cities is entirely different. The maximum possible development of the local water sources available would supply them for 10, or, at most, 15 years. Their distribution system, according to Mr. Dockweiler, is more than unsatisfactory, and their growth is already seriously hampered by the water situation. It would be advantageous for them, were San Francisco proper to "pull their chestnuts out of the fire." The rivalry between them and San Francisco is such that the latter would doubtless do nothing of the kind were the matter generally thus understood.

These communities face a serious situation. Their combined assessed valuation is only about $250 000 000. The ideal move would be for each section of Greater San Francisco-Oakland to acquire its

respective water system, extend and rebuild its distribution system, as the case may be, pool the local water supplies, and join in bringing in a common distant supply when necessary. Unfortunately, the local political and social situation makes such an arrangement improbable.

It is essential to realize that the water problem of Greater San Francisco-Oakland is very much more one for the mainland cities, than for San Francisco proper.

Marin County and North Bay Region.—What has been said of the East Bay cities is true of Marin County and the North Bay region, except on a much smaller scale.

VI.—Efforts Toward Municipal Ownership.

San Francisco, Oakland, and Denver are the only large cities in the United States which do not own and operate their water-works. According to present California laws, rates, extensions, financing, and service of all public utilities, outside of incorporated municipalities, are regulated by the State Railroad Commission. In November, 1914, the State will vote on the proposal to put all public utilities under the exclusive control of the Railroad Commission, and it is generally believed the change will be made.

Since 1880 municipal councils or supervisors have regulated, practically as to rates only, their public utilities. As elsewhere, this has been unsatisfactory, and at various times more or less serious steps have been taken to acquire or construct local municipal water systems.

San Francisco Proper.—The City had reports made by the late Gen. B. S. Alexander, Corps of Engineers, U. S. A., and the late George Davidson, Hon. M. Am. Soc. C. E., then of the U. S. Geodetic Survey, jointly (December, 1871); the late T. R. Scowden, M. Am. Soc. C. E. (May, 1874); and the late G. H. Mendell, M. Am. Soc. C. E., Col., Corps of Engineers, U. S. A. (1876-77). After the Mendell report the City officials tentatively offered $11 000 000 for the Spring Valley Water-Works' properties on the peninsula and the Company asked $12 500 000. The negotiations were dropped at this point.

On January 8th, 1900, a new charter went into effect, in which was declared the purpose of gradually acquiring the City's public utilities. Under it, the City Engineer at that time, C. E. Grunsky, M. Am. Soc. C. E., prepared plans for an entirely independent system, taking water from the Tuolumne River in the Sierra Nevada Moun-

tains, generally known as the Hetch Hetchy project. He estimated the cost of bringing 60 000 000 gal. per day to the Bay at $31 603 330 and the distribution system, at $8 807 000. After 12 years of effort on the part of the City the necessary rights of way, under severe restrictions, have recently (December, 1913) been granted by Congress in the passage of the so-called Raker Bill. On January 5th, 1914, the City officially accepted the grant and the conditions it carries.

In 1909 the voters passed on an alternative proposition for a bond issue of $45 000 000 to build the Hetch Hetchy project, including a distribution system, or an issue of $65 000 000 to bring in Hetch Hetchy water and buy the Spring Valley Water Company properties for $35 000 000. The latter lacked about 1 100 votes of the necessary two-thirds majority, and the former was carried.

On May 11th, 1908, the Secretary of the Interior at that time, the Hon. James R. Garfield, issued what is known as the "Garfield Permit," granting the rights of way requested by the City in the Yosemite National Park. This required the development of other portions of the project before utilizing the Hetch Hetchy Valley, a wonderful canyon of rugged beauty second only to the Yosemite Valley. On account of many protests, Secretary Ballinger, in March, 1909, acting on the advice of a committee consisting of Dr. George Otis Smith, Director of the U. S. Geological Survey, and L. C. Hill and E. G. Hopson, Members, Am. Soc. C. E., Supervising Engineers of the U. S. Reclamation Service, ordered the City to show cause why the Hetch Hetchy Valley should not be eliminated from the permit. A board of Army Engineers was appointed by President Taft, to assist and advise the Secretary in the matter. The members were Col. John Biddle, Lieut.-Col. Harry Taylor, and Col. Spencer Cosby, Members, Am. Soc. C. E. The appointment of this Board was probably due to President Taft's well-known penchant for the Army. Doubtless, none of these military engineers would urge his qualifications by training or experience as an expert in irrigation, hydroelectric, or municipal water supply matters.

At the request of the City Engineer, Marsden Manson, M. Am. Soc. C. E., Mr. John R. Freeman was called in by San Francisco to assist it in presenting its case. He associated with him eleven other members of the Society and President J. C. Branner of Stanford Uni-

versity. Afterward, M. M. O'Shaughnessy, M. Am. Soc. C. E., succeeded Mr. Manson (September, 1913), and shouldered much of the responsibility. Mr. Freeman fundamentally changed the Hetch Hetchy project as theretofore outlined.

The hearing was held by the Secretary of the Interior, the Hon. W. L. Fisher, on November 25th to 30th, 1912, and it is greatly to be regretted that the proceedings have not yet been published. The Army Board presented a report, largely the work of H. H. Wadsworth, M. Am. Soc. C. E., Assistant Engineer, Rivers and Harbors Branch, Corps of Engineers, U. S. A., on February 19th, 1913. On March 1st, Secretary Fisher decided that he was not authorized to act, and recommended Congressional action. His successor, Secretary F. K. Lane, concurred in this. Hence, there followed the introduction and passage of the Raker Bill.

On December 31st, 1913, the City of San Francisco filed suit to condemn the properties of the Spring Valley Water Company. A year before, tentative negotiations had been broken off, with the Company asking $37 500 000 and the City offering $37 000 000.

East Bay Cities.—On the mainland, Oakland and Richmond are the only communities which have made definite efforts to acquire water systems and supplies.

Oakland.—In 1902 Rudolph Hering, M. Am. Soc. C. E., made a report for a Citizens' Committee on the water supply for Oakland. He advised that 15 000 000 gal. per day of underground water could be obtained from the delta cones of Alameda Creek. In 1903, Desmond FitzGerald, Past-President, Am. Soc. C. E., reported to the same committee, and the next year a Board of Engineers, consisting of A. M. Hunt and J. M. Howells, Members, Am. Soc. C. E., and Mr. F. C. Turner, reported to the Oakland authorities on the adequacy of the Bay Cities Water Company's project for a local surface water supply from near Mt. Hamilton. A bond issue for taking over and building this project was defeated by the voters in December, 1904.

In the same year the rates of the Contra Costa Water Company (since absorbed by the Peoples Water Company), in Oakland, were fixed by the City authorities at a point so low that the Company appealed to the Courts. The result was the same as in San Francisco, namely, expensive litigation still pending and inadequate extensions of the system.

In 1910 litigation was begun between the Peoples Water Company and the City of Berkeley, but it was soon compromised by minor concessions on the part of the Company.

The formation of a municipal water district is now being considered.

The Richmond Municipal Water District.—This district was formed by an election held December 3d, 1912, covering the region bounded on the south by the Alameda-Contra Costa County Line and on the east and north by an irregular line approximating the eastern edge of the San Pablo Creek water-shed. It contains 56 sq. miles, of which 40 sq. miles, are below the 200-ft. contour. The population on June 30th, 1913, was estimated at 15 585, of which 13 575 were in the City of Richmond. The present assessed value in the district is $19 400 000.

The present daily water consumption is 3 400 000 gal., equal to 194 gal. daily per capita, of which 123 gal. are used in manufacturing and industrial pursuits. All water is pumped from underground sources, and almost exactly one-half is from private wells, the draft from which has about reached the available limit. The future per capita use will probably increase to 230, and then gradually drop to 150 gal. per day when the population reaches 150 000. Such large per capita consumption is expected because the industrial growth for many years will exceed the residence and business increase. There is no adequate fire protection.

The engineers of the district, Mr. P. A. Haviland, M. Dozier, Jr., M. Am. Soc. C. E., and F. H. Tibbetts, Assoc. M. Am. Soc. C. E., of San Francisco, have reported exhaustively on two water sources available. These are, a reservoir on San Pablo Creek, and by pumping from the Sacramento River at Toland's Landing, with an emergency intake at Antioch. The safe draft of 12 000 000 gal. daily can be secured from San Pablo Creek at a cost of about $3 500 000, and a like quantity brought from the Sacramento River, including a mechanical filtration plant, for from $4 000 000 to $5 600 000, depending on the conduit route and the materials used. The immediate development of a filtered river supply for 6 000 000 gal. daily, at a cost of from $2 300 000 to $4 100 000, depending on location and construction details desired, was recommended. The District's Board has adopted this recommendation and is arranging with

the towns *en route*, Martinez, Port Costa, and Pinole, to join in the project, and a bond election to provide funds will at once be held.

The present rates are from 30 to 35 cents per 1 000 gal., and it is estimated that, if the proposed plans are carried out, the cost will be from 20 to 25 cents, reducing in a few years, through large consumption, to 15 cents.

Marin Municipal Water District.—The present per capita water consumption is 85 gal. daily, a very small quantity for the class of urban development served. This is partly due to the actual inability of obtaining water as desired, but, chiefly to the high cost, which is 50 cents per 1 000 gal. and upward. Such high rates are in considerable measure due to the topography of the country. Early in 1912 the Marin Municipal Water District was formed, covering 115 sq. miles, in which the assessed valuation is $13 000 000. In November of that year A. R. Baker, Assoc. M. Am. Soc. C. E., was appointed Engineer, and M. M. O'Shaughnessy and Edwin Duryea, Jr., Members, Am. Soc. C. E., Engineering Advisers. It is understood that steps will immediately be taken to carry out Mr. Baker's recommendations. These are:

1.—Build a dam on Lagunitas Creek about 4 miles below the present reservoir of the Marin Water and Power Company (Lake Lagunitas), on the northern slope of Mt. Tamalpais, and create a storage capacity of 6 400 000 000 gal., or 19 700 acre-ft. In this way a daily supply of 5 000 000 gal. is expected to be obtained. The water will be taken around the shoulder of the mountain in tunnels and pipes, and delivered wholesale to the various localities from a system of main supply pipes. The total estimated cost of this first installation is $2 000 000.

2.—Take over the properties of the Marin Water and Power Company and the North Coast Water Company, and in time develop all their resources, together with the proposed new work, to the full capacity, estimated at 14 000 000 gal. daily, at a total cost of about $5 000 000.

3.—An alternative plan, not recommended, of pumping water from wells alongside the Russian River, about 25 miles from the north end of the district, and conveying it through Santa Rosa, Petaluma, and other communities *en route*, each of which would probably become customers. The cost for installations of 5 000 000 and 15 000 000 gal.

daily, from this source, would be very nearly equal to those respectively recommended.

The local distribution systems, like those of the East Bay cities, are without broad design, and leave very much to be desired, particularly in the matter of fire protection and permanency of materials.

VII.—Essential Factors Affecting a Choice of Distant Water-Supply Sources.

Far-sighted planning for a large additional supply should consider and provide for the water needs, not of the East Bay region, nor of San Francisco proper, but of Greater San Francisco-Oakland as a whole. Because there is a strong sentiment in the East Bay cities against joining with San Francisco in any political or financial enterprise, at least as long as San Francisco has a predominating influence, and because of a rapidly growing realization that in many ways the two sides of the Bay are competitive, it is possible the East Bay cities will act independently.

Quantities Required for Various Dates.—The additional quantities needed by the several portions of and for all Greater San Francisco-Oakland are about as given in Table 2.

TABLE 2.

	1920	1940	1960	1980	2000
San Francisco Proper and Southern Peninsula	10	50	100	155	200
East Bay Region	12	85	215	350	480
Marin County and North Bay Region	2	5	12	23	37
Totals	24	140	327	528	717

It will probably be economically desirable to obtain from outside sources about the quantities given in Table 3.

TABLE 3.

	1920	1940	1960	1980	2000
San Francisco Proper and Southern Peninsula	0	0	0	5	50
East Bay Region	3	46	178	312	443
Marin County and North Bay Region	0	0	1	9	23
Totals	3	46	179	326	516

Doubtless it would be profitable, certainly desirable, and at least a very nice thing, to furnish water to the urban communities along or close to the conduit or aqueduct line, and to the mutual interests of all concerned that excess water be used for irrigation along that line, until needed in Greater San Francisco-Oakland. Under existing laws, this would endanger a servitude being placed on such excess water. It must be remembered, however, that even the State Constitution can very easily be changed—especially with the initiative, referendum, and recall in effect.

Discounted Present Value Only Fair Method of Comparison.—Distant sources have the serious disadvantage that, as single-conduit installations cost much less than double or triple lines of equal capacity, larger and more expensive works than present needs demand are economically required; and interest charges on initial investments mount up startlingly. Those sources which are adaptable to gradual development have a marked advantage. Greater San Francisco-Oakland may obtain water, satisfactory as to quantity and quality, from so many different sources, that, to make fair comparisons, it is necessary to outline the expenditures of all kinds, construction, operation, maintenance, depreciation and renewals and incidental incomes, at specific dates, and reduce them to present values, using probable interest rates.

Location of Local Storage.—Another disadvantage of distant water sources is increased likelihood of damage to the conveying conduit or aqueduct. As the distance increases, other things being equal, not only is the risk of sudden interruption greater, but also the time required to make repairs, the cost of patroling, and of up-keep. To avert serious consequences in case of interruption, it is necessary to provide large, near-by storage facilities. The greater the risk the more storage is required.

Fortunately, San Francisco proper has Crystal Springs Reservoir on the Peninsula (capacity 19 300 000 000 gal., or 59 200 acre-ft.), and in a few years will have the Calaveras Reservoir (capacity 53 000 000 000 gal., or 162 000 acre-ft.), on the east side of the Bay; the East Bay cities have Lake Chabot (capacity, 4 840 000 000 gal., or 14 800 acre-ft.), with the possibility of creating the larger Upper San Leandro Reservoir (capacity, 16 700 000 000 gal., or 51 270 acre-ft.), at a cost of $3 000 000. Marin District now has Lake Lagunitas and Phœnix Lake (capacity, 382 000 000 gal., or 1 186 acre-ft.), and probably within a few years

another larger storage basin which the Marin Municipal Water District plans to build (ultimate capacity, 6 400 000 000 gal., or 19 700 acre-ft.). The location and elevation of these local storage facilities have an important bearing on the selection of a distant source.

Earthquake Zones and Faults.—The seismic disturbances which have occurred thus far in California have been *tremblors,* and not very serious matters, provided—but only provided—their likelihood is frankly recognized, the facts concerning their occurrence thoroughly ascertained, and methods to circumvent them carefully and systematically worked out. President J. C. Branner, the eminent geologist of Stanford University, urges scientific, sane, and calm study of the "active faults" in the earth's crust in California. When these are located it will be possible to build so that when future earth slips occur, the damage will be negligible. If the wriggling line of the "1906 Fault" had been accurately known, the Spring Valley Water Company would not have had its pipe line laid over it, the water supply would not have been cut off, and the disastrous fire would not have happened.

Crossings of Waterways, Bay, and Mt. Diablo Range.—There are serious natural obstacles to bringing water from distant sources into San Francisco. With the Sierra Nevadas to the east there is the wide San Joaquin Valley, the River, and the Mt. Diablo Range. The first of these prohibits a gravity line, except by using very long inverted siphons under high pressure, the last necessitating pumping against high head over the lowest pass, or long tunneling in very bad ground crossed by "active faults." To bring water from the northern Sierra Nevadas, requires crossing the narrower but equally low Sacramento Valley, the River, and the Carquinez Straits. All Sierra Nevada sources necessitate taking the water for San Francisco proper across San Francisco Bay. With southern sources Marin County can only be supplied through a tunnel more than a mile long under the Golden Gate. The water here is 300 ft. deep, and the crossing would probably be somewhat similar to the one under the Hudson River at Storm King on the Catskill Aqueduct.

Many local engineers fear the difficulties of tunneling the Mt. Diablo Range, regarding it as a formation exceedingly difficult and expensive to pierce. Mr. Freeman, on the other hand, regards the difficulties it presents as much less than those of the Carquinez Straits

crossing, which latter he regards as offering such tremendous obstacles alone as to dismiss immediately from serious consideration all sources which would involve it.

Conservation.—There can be no doubt that supplying municipal needs is the highest use to which water can possibly be applied. That is by no means saying that other useful requirements should not be considered at all until such highest use is fully satisfied. All needs of a region should be considered in relation to all supplies, and adjustment made to secure the maximum total economic benefits to society. This is so obvious that taking the trouble to state it seems waste of time. Nevertheless, it is usually adroitly ignored and avoided for the reason that the greatest total benefits to society may mean individual lesser benefit or greater cost to a powerful interest. The total benefits and costs are rarely divided in like proportions. To this fact is due conservation and the conservation policy—now generally accepted in theory and almost as generally sought to be avoided in individual cases.

In California, as in all the Far West, because of climatic conditions, water is more important than fertile soil and strategic position. The water resources of the Central Valley and the northwestern part of California, sooner or later, will be so conserved and apportioned as to produce the maximum possible benefits. No use can be so high as to interfere permanently with, or prevent, such a result. Unfortunately, no definite steps have yet been taken toward such end, and this fact alone seems to justify the National conservation movement. It is extremely unsafe, however, for any interest to assume that such indifference will continue. The world moves and, in the Far West, it moves rapidly. Indeed, it is startling to look back only 10 years and see how the viewpoint of the general public has changed with regard to the exploitation of natural resources and how it has already been crystallized into fundamental changes in legislation. Any person, corporation, or municipality is most short-sighted to make investments without weighing carefully the probable changes in public opinion and consequent legislation, designed to produce a higher degree of intrinsic justice—at least for the period necessary to "unload" successfully.

Furthermore, municipalities are but mutual corporations. In Wisconsin, municipally owned public utilities have for years been regu-

lated as to service, rates, and accounting, exactly like privately owned and operated utilities; and Wisconsin's lead in such matters has been closely followed in the Progressive West.

Irrigation.—Next to municipal use comes irrigation. In discussing the growth within California, the agricultural history of the Central Valley has been touched on and the radical changes at present under way noted. The American, and particularly the western, characteristic attitude toward experience elsewhere, resulted in irrigationists having to learn by bitter experience the wisdom of the United States Reclamation Service's standard of satisfactory irrigation water supply. This standard is that all water users have ample water at all times when it can be used profitably. It involves storing flood-waters for use when streams are very low or entirely dry during the latter part of the summer. The California Central Valley has been almost the last part of the United States to find itself in this regard.

The tremendous value of the climate and fertile soil in the great Central Valley is still not appreciated. The crop adaptation is marvelous. It is conservative to say that 20 years hence an acre of properly irrigated bare land anywhere in the Central Valley will be worth at least $500, and citrus land in the foot-hills from $600 to $800. It is equally conservative to say that in the central and southern parts of the San Joaquin Valley, particularly, unirrigated agricultural land will be worth not more than from $20 to a possible maximum of $50 per acre.

Power.—Next in order of use is the development of hydro-electric power. California stands near the head of all States in the quantity of developed water-power, and yet only a fraction of its possibilities has been utilized. The cheapest fuel is oil from local fields, now costing about 75 cents per bbl., wholesale, equivalent to coal at $3 per ton, and the supply is by no means limitless. The tremendous economic importance of the power resources of the State is obvious.

The difference between the ultimate value to society of a natural resource and the commercial value of a development of that resource must be understood. Local and general economic conditions have resulted in smaller hydro-electric concerns gladly disposing of their output in blocks at high load factors to the large companies for about ½ cent per kw-hr. One of the largest power corporations in the State, with hydro-electric plants scattered for miles along the western slope

of the Sierra Nevadas, transmission lines gridironing the central part of the State, and a 60% load factor, is generating more than 40% of its energy in steam plants.

Indeed, there is a general misconception of the commercial value of "water rights." Every decrease in cost and increase in efficiency of steam plants reduces the relative advantages of water power. The steam turbine is a notable example. The "value of water rights" has lowered tremendously in the past few years, because of rate and service regulating commissions. The tendency is to regard a water right, not as an asset, but as an exclusive license for self-constituted trustees to develop a public resource and operate it for the benefit of the public, on a percentage. This change of sentiment has practically wiped out the commercial value of a proposed project controlled by a "water filing." Companies doing a tremendous business and having ample credit can wisely carry out enormous developments and secure thereby very low costs per unit of installed capacity. The rapid development and constantly lowering cost of electric generating and transmitting equipment and the progress of the art in general are other important factors.

For example, one of the recent large projects in the State, now more than half completed, is a development in which water will be dropped through six power-houses and generate 165 000 h.p., with a total cost of $130 per installed kw. or $97 per h.p. Even this will be reduced, because water from the lowest plant will irrigate a large body of land and the State Utility Commission will certainly take irrigation water rentals into account in fixing power rates. Obviously, such developments affect the value of all hydro-electric plants and projects looking to San Francisco as a market, if not throughout the whole State.

This matter has been considered at length because the Army Board evaluated the "incidental power possibilities" of the Freeman Hetch Hetchy project—115 000 h.p.—at $51 100 000. This is more than $445 per h.p. On such a basis, the water rights of the power company's complete development, just mentioned, would have to be $37 425 000. A completed hydro-electric installation feeding into a completed transmission system supplying an actually developed market, is probably worth not more than $125 per h.p.—without such market, particularly, and the means of serving it, much less. Los

Angeles is discovering this in connection with its aqueduct power, as the city itself finds it impossible to use the hydro-electric power at a high average load factor.

Navigation.—The Sacramento River is technically navigable to Red Bluff and the San Joaquin River up to Firebaugh. Both streams are much used in the lower stretches—the former to Sacramento and the latter to Stockton. Beyond these latter points navigability is at present important chiefly on account of holding down local railway rates. Consequently, navigation in this case ranks below irrigation and power uses.

Obviously, any water storage which increases the natural flow at low-water stages improves navigation. Diversion from tributaries and upper reaches of main streams for irrigation is usually, though not always, conflicting. Diversion from higher areas, entirely outside the water-shed, as a supply for Greater San Francisco-Oakland, would always be conflicting, though, with some sources, with very small low-water flow, negligibly so.

In the Sacramento Valley there are several available storage possibilities by which the flow of the Sacramento River at Red Bluff could be increased by 500 sec-ft. for 4 months, and numerous ones to provide such increase below Marysville. In this way, navigation interests could easily and relatively cheaply be protected to permit diverting enough water from any part of the Valley drainage to supply Greater San Francisco-Oakland.

In the San Joaquin Valley the situation is not so fortunate. There the water supply is so deficient, except possibly in the northern end, that the use of storage opportunities in this way cannot be afforded.

Flood Protection.—Last of all—so far as storage is concerned—comes flood protection. The 1 750 000 acres flooded in the Central Valley must always be protected by levee systems almost exclusively. Of course, storage for almost any use would help.

Senator Newlands, the author of the Reclamation Service Act, has recently introduced a bill in Congress providing $60 000 000 per year for 10 consecutive years for flood protection, of which one-twelfth is for the Sacramento and San Joaquin Rivers, and the California Reclamation Board has outlined a $50 000 000 project for the Sacramento Valley. Spending such tremendous sums for reclaiming

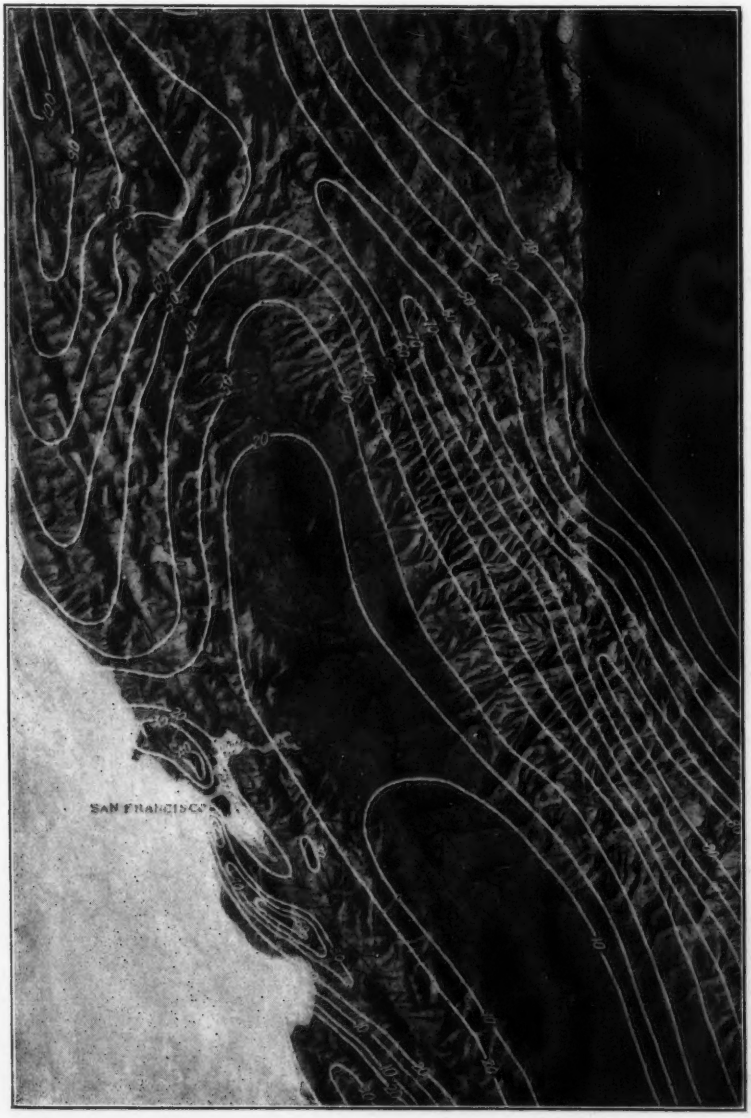
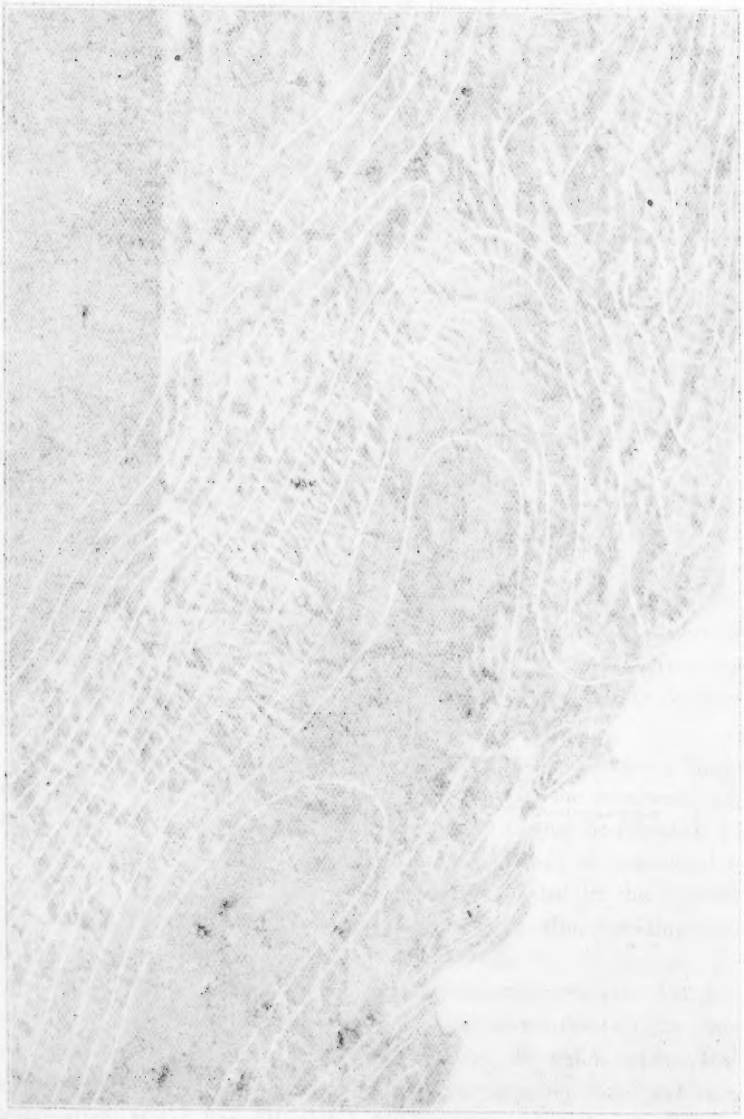


FIG. 15.—RELIEF MAP, SHOWING PRECIPITATION OVER CENTRAL CALIFORNIA.



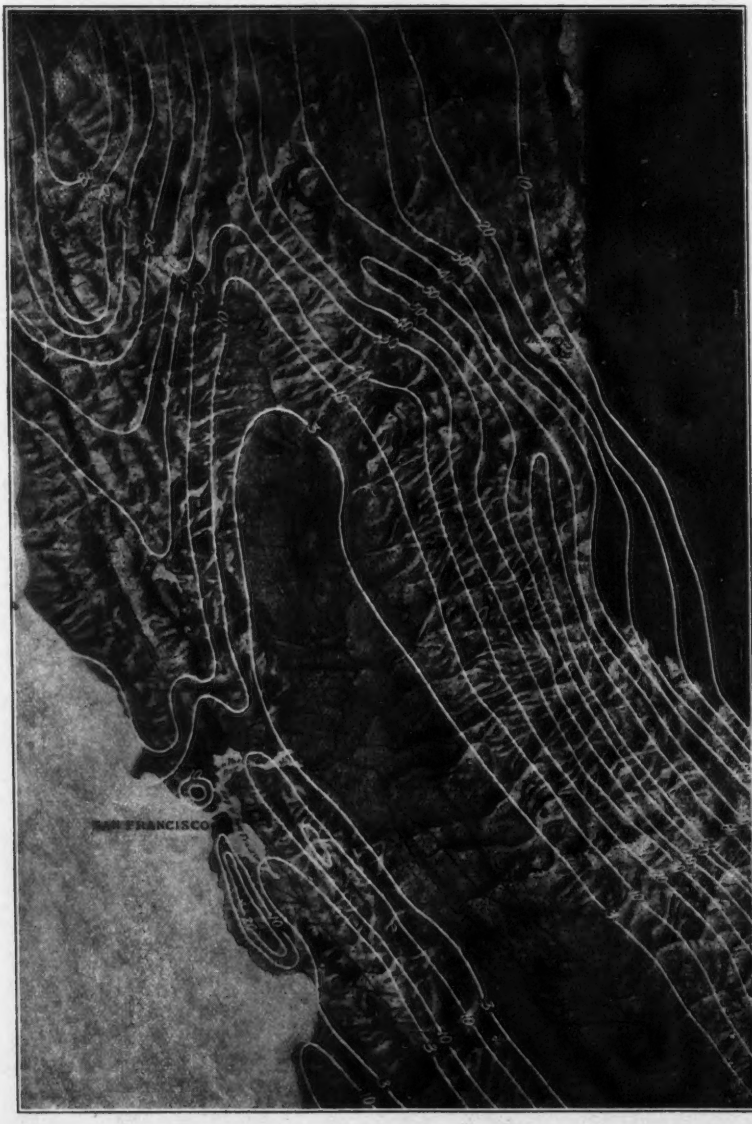


FIG. 16.—RELIEF MAP, SHOWING RUN-OFF OVER CENTRAL CALIFORNIA.

bottom-lands and neglecting to prevent waste of such citrus lands through unwise distribution of water resources is foolish.

All Uses Interwoven.—All these utilizations of the Central Valley's water resources are inter-dependent. To secure the greatest total benefits to society offers no serious problems in statecraft or engineering —only an intelligent general plan. In some sections there will then be water to waste, in others a deficiency. In any event, all comers, no matter of how high or of how low degree, or of what precedence their claims, will sooner or later be made to conform with such a general plan.

California Precipitation and Run-Off.—Figs. 15 and 16 show the probable mean precipitation and run-off curves for the State, based on all the data yet collected.

It is more than unsafe to design hydraulic works in California from mean annual figures, because of dry cycles often 6 and 7 years long. The precipitation, speaking generally, is due to storm centers coming over the Pacific Ocean and passing on to the east. When the center enters the United States near Puget Sound, practically no precipitation results in the southern half of the State, but when the path lies farther south, say, at San Francisco, the entire State receives rain. For some reason these storms often pass well to the north for several successive years. Each year in addition, there are usually three or four relatively unimportant storm centers which pass over or near San Diego. Nevertheless, it is said that the records, kept by the Franciscan Fathers at the Mission in Santa Barbara for more than 130 years, show 1 year with absolutely no measurable precipitation.

As a consequence, in the southern end of California all significant storms are general, and in dry years so gentle and distributed that the percentage of run-off is almost incredibly small. Going northward, such conditions are less noticeable, but, throughout the entire State, especially as far north as Sacramento, long dry-year cycles frequently occur.

There are few run-off data in California prior to 1901, when the State and the U. S. Geological Survey began co-operating in systematic stream gauging. There are shorter cycles of dryer years before this period, however. This fact must be kept in mind in connection with the run-off curves given. For example, one would make a grievous

mistake by using all the stream gaugings after 1902 in Fig. 17, and assuming them to be typical. There were few reservoir records prior to 1901 in the Sierras, like those at Sweetwater and Cuyamaca.

Irrigation Water Requirements.—In determining the excess, if any, of available water above the needs for irrigation in any territory, there are the two factors, irrigable area and the local duty of water. The first is simply measured, except that in some cases the greatest utilization of a large region's water resources may require modifying the natural limits, such as water-sheds.

The local duty of water, however, is as yet a matter of judgment. It varies with soil types, present and future crops, drainage, character of supply, metered or unmetered service, and extent of cultivation. Very little Middle and Northern California data are enlightening, because very few supplies are really what they should be. Water rentals on a quantity basis are the rare exception. The almost universal disposition of irrigators to use too much water is well known. Few supplies are dependable, particularly during the latter part of the irrigating season, when streams are at the lowest. This results in using excessive quantities during times of plenty, to store water in the soil, a wasteful plan, unsatisfactory in many ways. Thus water-logging and temporarily ruining large areas is the general experience in almost all western irrigation projects. Averages of current practice, therefore, are thoroughly unsafe guides; the extreme limits are too far apart.

The various State Agricultural Experiment Stations and the U. S. Department of Agriculture have made numerous experiments to check such collected data. Unfortunately, these data do not have great practical value, just as the results of acceptance trial tests on steam-power plants are not safe bases for estimating output cost per horse-power per annum.

An important economic factor is the degree of soil cultivation. When the Sweetwater Reservoir was empty and the precipitation very small, citrus trees near San Diego were kept alive with only 3½ in. of irrigation water per annum, but at a cultivation cost normally prohibitive. Dry farming could be practiced successfully over vast areas where it is economically unjustified under conditions now and for a long time obtaining. The two greatest problems in America are to stop the disproportionate growth of cities and to improve the labor situation on the farm. Assuming that 1 acre-ft. of water per

annum and a cultivation cost of $25 per acre will produce as much as 3 acre-ft. of water and a cultivation cost of $5 per acre, is the water saving justifiable? Except where the water supply must be extended

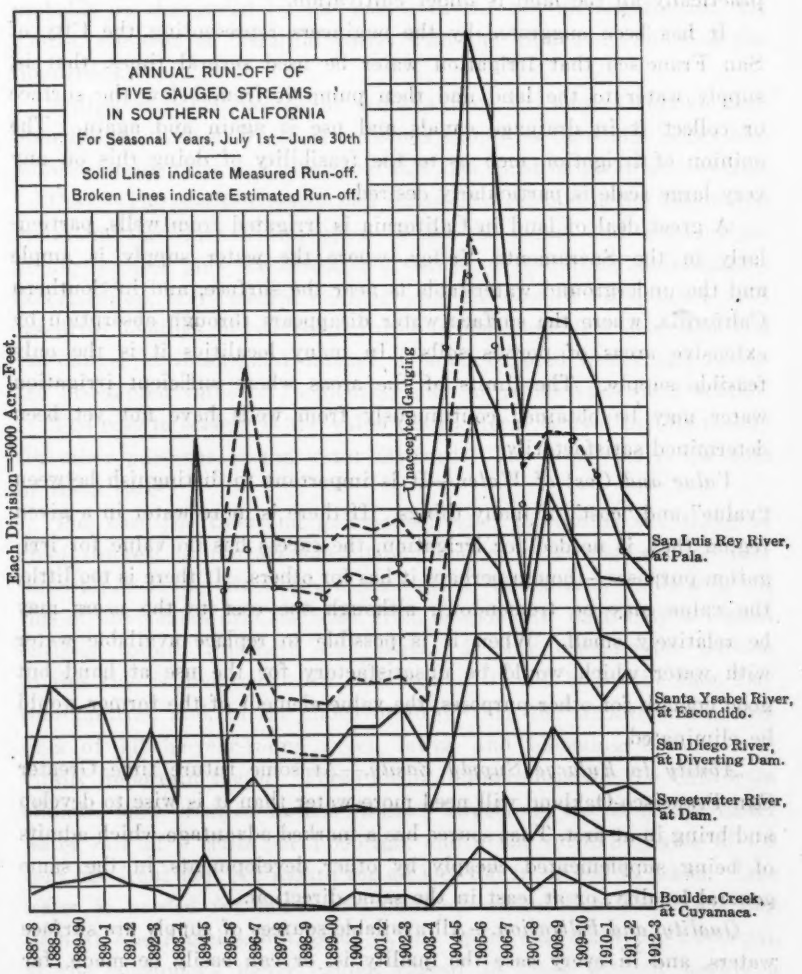


FIG. 17.

over exceptionally valuable land, obviously it is not. Experimental or theoretical and economical minimum water duties are very different things.

Many irrigation engineers believe the only safe guide is the actual water use over large successfully cultivated areas where conditions are essentially similar, where the water supply is metered, and where practically all the land is under cultivation.

It has been suggested by the engineers representing the City of San Francisco that irrigation water be used several times, that is, supply water to the land and then pump it from below the surface or collect it in drainage canals and use it again and again. The opinion of irrigation men as to the feasibility of doing this on any very large scale is particularly desired.

A great deal of land in California is irrigated from wells, particularly in the Sacramento Valley, where the water supply is ample and the underground water-table is near the surface, and in Southern California, where the surface water disappears through absorption by extensive areas of porous soils. In many localities it is the only feasible supply. The limits of the areas where sufficient irrigation water may be obtained continuously from wells have not yet been determined satisfactorily.

Value and Cost of Water.—It is important to distinguish between "value" and "cost" of many things. If there is more water in a given region than is needed for irrigation, the excess has no value for irrigation purposes—though perhaps it has for others. If there is too little, the value may be tremendous, although the cost to the users may be relatively small. When it is possible to replace available water with water which would be unsatisfactory for the use at hand but good enough for other purposes, the value element of the former would be eliminated.

Ability to Enlarge Supply Easily.—At some future time Greater San Francisco-Oakland will need more water than it is wise to develop and bring in at first. That source has a marked advantage which admits of being supplemented cheaply by other developments in the same general locality, or at least in the same direction.

Quality, and Filtration.—All available sources of supply are surface waters, and in every case the quality is, or can easily be made, far above what is generally regarded as satisfactory for large cities. Some are better in this regard, others in that; some will require immediate filtration, with others it would be unnecessary. The disposition is to insist more and more on treatment of all surface supplies, even when long

storage in reservoirs is possible. Quality standards in everything are rising, and filtration to remove coloring matter and minute organisms, even when, for hygienic reasons alone, it would be unnecessary, will probably be demanded by the public in the not distant future. Of course, the cost of filtration varies with the kind and extent of the impurities.

Sentiment.—The people of Greater San Francisco-Oakland have been taught to regard a Sierra Nevada supply as markedly better and more desirable than any other, and this fact must be given due weight.

Cost.—The cost to Greater San Francisco-Oakland is the discounted values of the expenditures required for the rights and construction of any project plus the discounted values of operation, maintenance, deterioration, and renewal charges, plus the cost of antagonistic and resentful feelings of all communities which may feel despoiled and the danger of intentional damage to much-exposed works, and minus the discounted value of incidentals such as power and water rentals.

The cost to society is the value (as the term is used in this paper) for other purposes of the water diverted. The total cost is the sum of these two.

The cost per thousand gallons delivered to him is the thing of chief interest to the average citizen, and of particular and peculiar importance to the East Bay communities. As already explained, the tide of newcomers into the State from east of Salt Lake City is almost entirely toward Southern California. If any considerable portion of this tide is diverted to the Bay regions, it will be by the East Bay communities, and, in the endeavor to do this, the striking attractiveness of all-the-year-round green lawns and blooming flowers is a vitally important factor. These media, all accentuating the California climate, have resulted in Los Angeles largely by reason of the meter water rate of 9 cents per 1 000 gal. and a flat sprinkling rate of 0.9 cent per front foot per month for all lots 150 ft. or less in depth. Less water is needed for lawns and gardens around San Francisco Bay than at Los Angeles, but, even so, the upper cost limit per 1 000 gal. at the consumer's meter cannot exceed 15 cents without having a serious effect on the area of lawn in the East Bay district.

It is a luxurious thing to have a high mountain water supply, but it is not an obvious point in competing for the Eastern and Middle West newcomers to the State. In this day and generation, to urge a

pure water supply is about as impressive as to dilate on satisfactory police protection, or paved streets—these things are assumed as a matter of course. Green lawns and blooming flowers the year round, and especially in winter, made possible by California's distinctive climate, is constantly impressive to to-day's tourists and to-morrow's fellow-citizens, and these features must be practically universal to compare at all favorably with the southland.

It is a simple matter to take a base rate of 15 cents per 1 000 gal., meter measurement, the present or prospective consumption at any given time, the cost of operation, taxes, etc., and, with any assumed rate of interest, ascertain the capital investment which may be expended on water supplies without jeopardizing this matter of lawn development. Suffice it to say that any proposed water development on a large scale, which would make water cost more than 50% greater than the rates obtaining in Los Angeles, is practically beyond the means of the East Bay cities. This point is of little importance in San Francisco, because there are notably few and small lawns, and even with free water from now on, that city never could be made comparable in that way with Los Angeles.

VIII.—Possible Distant Sources of Supply.

The following additional sources of supply have been suggested: Eel River, Putah Creek, and Clear Lake and Cache Creek, from the northern Coast Range region; Sacramento and San Joaquin Rivers, from the delta region of the Central Valley; and McCloud River, Indian Creek, Feather, Yuba, and American Rivers, Lake Tahoe, Mokelumne, Stanislaus, and Tuolumne Rivers, from the Sierra Nevadas.

A.—Coast Range Sources.

The Coast Range sources are shown on the map, Fig. 18. The water is not needed locally for irrigation. The length of conduit is much less than that for any of the Sierra Nevada sources, and, as a supply for only the East Bay cities and Marin County, is apparently well worth more careful consideration than it has received. In the construction of the project, the Carquinez Straits crossing should be made sufficiently large to serve as well for the maximum possible additional supplies, such as the McCloud, the Indian Valley, and the Feather River projects, when any or all of these may be needed.

FIG. 18.—RELIEF MAP, SHOWING POSSIBLE COAST RANGE SOURCES OF SUPPLY.

FIG. 19.—RELIEF MAP, SHOWING POSSIBLE NORTHERN SOURCES OF SUPPLY.

SAN FRANCISCO

Eel River.—Above the Gravelly Valley Reservoir site, where 180 000 acre-ft. capacity could be created by a dam 140 ft. high, is a drainage area of 326.5 sq. miles. The safe draft is estimated at 180 000 000 gal. daily, or, with a conduit of 225 000 000 gal. daily capacity, and using the large terminal storage in Greater San Francisco-Oakland, 200 000 000 gal. daily. The conduit would be so near the hydraulic grade line that reinforced concrete pipe could be used for much of the distance. The elevation of the intake is 1 000 ft., and the length of the conduit to San Francisco, *via* Golden Gate, 125 miles. No cost estimates are available.

Putah Creek.—Putah Creek is the closest mountain source. With the excellent Monticello and Guenoc Reservoir possibilities, a safe draft of about 150 000 000 gal. daily could be secured. The conduit could follow closely the hydraulic grade line to Carquinez Straits and be 60 miles long to Lake Chabot, back of Oakland. The water should all be used locally for irrigation.

Cache Creek and Clear Lake.—Within the possible limits of lake-level regulation, 600 000 acre-ft. of storage could be obtained in Clear Lake, with which a safe draft of about 195 000 000 gal. daily would be possible. This water would not be of the best, and it should all be used for irrigating tributary valley and foot-hill land.

B.—Delta Sources.

The Delta sources are shown on the map, Fig. 20.

San Joaquin River.—The low-water flow of the San Joaquin River is so small that an intake would have to be constructed where inflow from the Sacramento River could be assured, and yet be high enough to avoid salt water from the Bay. Mr. Wadsworth suggests Clifton Court on the Old River. The supply would have to be filtered. The Sacramento River water is clearer, of better quality, softer, has less mineral matter, and a much less seasonal variation.

Sacramento River.—At the mouth of the Sacramento River the water becomes slightly brackish late in the seasons of low run-off. At Toland's Landing, 5 miles above, it is always fresh. In the future, increased irrigation use will tend to reduce the flow during the early part of the season, but will probably slightly increase the low-water flow, though storage for power development and other purposes will materially increase the low flow. It is highly improbable that sea

water will ever get materially farther up stream than at present. Mr. Allen Hazen reported in 1912 that the "Sacramento River water be filtered so as to produce a good potable water at all seasons of the year, removing the turbidity, color, and results of sewage pollution." The resulting hardness would be from 56 to 62%, or about two-thirds that of Washington, Cleveland, Chicago, Pittsburgh, Cincinnati, or Louisville, and 80% more than New York or Philadelphia.

The filtration plant and an emergency intake would be near Antioch, as planned for the Richmond Water District. The total length of the line recommended by Mr. Hazen, starting about 8 miles farther up the river, at Rio Vista, running thence directly to filtration works at Antioch, and thence to Walnut Creek, East Oakland, Alameda, and across the Bay to Potrero Point, San Francisco, is about 54 miles.

Estimates on this project were with a $3 wage scale for an 8-hour day and present railway freight charges on material from the East. Taking dates for installations of 133 333 333 gal. daily, except tunnels built originally for 400 000 000 gal. daily capacity, as 1914-18, 1936, and 1975, respectively, interest at 4½%, and assuming that all plants would begin running at capacity immediately on installation, at a cost of $17 per 1 000 000 gal. for operation, maintenance, depreciation, and renewals of plant, the present value of the cost of this source is estimated by Mr. Wadsworth as $51 700 000.

Doubtless, no one would seriously consider installing works of 133 333 333 gal. daily capacity for a community now fully supplied and using only 70 000 000 gal. daily, nor begin to operate them to full capacity immediately on completion. Caution must be used, therefore, with this figure of $51 700 000.

The water diverted would have no value for any other purpose whatever. No power could be generated, and practically no water sold *en route*.

C.—Sierra Sources.

The Sierra sources are shown on the map, Fig. 21.

McCloud River.—The McCloud River rises on the south side of Mt. Shasta, and, joining with the Pit River, forms the principal tributary of the Upper Sacramento River. It has a remarkably steady flow for California, and a minimum discharge of 1 200 sec-ft., or 770 000 000 gal. daily—amply sufficient for all possible needs. The quality is excel-

FIG. 20.—RELIEF MAP, SHOWING POSSIBLE DELTA SOURCES OF SUPPLY.

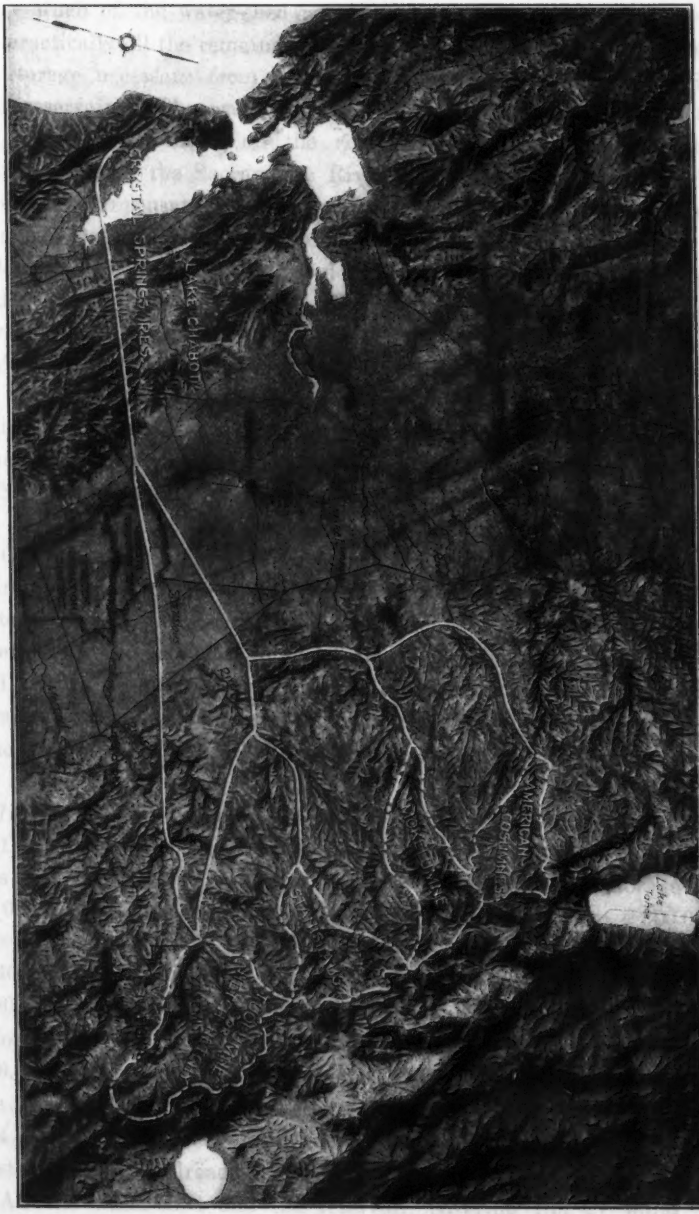


FIG. 21.—RELIEF MAP, SHOWING POSSIBLE SOUTHERN SOURCES OF SUPPLY.

lent. Much of the water-shed is in the Mt. Shasta National Forest, and practically all the remainder, 400 000 acres, is offered for $3 000 000. All storage necessary from hygienic considerations could be had in local reservoirs. The water is not needed for irrigation, and there are storage possibilities on the river itself and elsewhere in the drainage area of the Sacramento River above Red Bluff which would more than compensate for any purpose for any quantity Greater San Francisco-Oakland could wish to divert during low-water stages. It is doubtful whether the City would ever have to do anything of the kind—certainly not for years. The comparison of this with other Sierra Nevada sources is thus practically one of cost only.

An aqueduct to Lake Chabot, in Oakland, would be 212 miles long. Mr. Wadsworth's estimate for the discounted present value of an initial installation of 260 000 000 gal. daily, 1914-20, and an additional 140 000 000 gal. daily, in 1973, is $52 500 000.

No power would be developed along the aqueduct line, but, on the project and within the controlled tributary water-shed, about 130 000 h.p. can be developed at attractive costs. The conduits, almost entirely of gravity type and of reinforced concrete, would be along the western edge of the Sacramento Valley for the entire length of the latter, and would pass close to many rapidly growing towns and irrigable land having the highest crop and fruit adaptation in the State. All excess water could doubtless be sold along the way from the very start, although not at prices fully proportionate to the cost of the aqueduct and reservoir system.

Indian Creek.—Indian Creek is a branch of the North Fork of the Feather River, with one of the best and most economical reservoir sites in America. A dam 120 ft. high would create a storage of 600 000 acre-ft. and one 220 ft. high and 900 ft. long on top, 2 300 000 acre-ft. (a capacity of 445 000 acre-ft. is equivalent to a yearly flow of 400 000 000 gal. daily). The flow line would have an elevation of 3 680 ft., and the area would be 11 500 acres. The mean annual evaporation is about 3½ ft. The run-off has been gauged from January, 1906, to date, except from January 1st, 1909, to September 10th, 1911. The average annual discharge seems to be about 480 000 acre-ft. The dam site is 4 miles, by good automobile road, from Keddie on the Western Pacific Railroad.

All the water-shed and the reservoir, covering 733 sq. miles, is

in the Plumas National Forest, and, except a total of 30 000 acres, which would cost about $1 100 000, is public land. There would be no interference with irrigation or navigation, and fewer legal entanglements than with any other Sierra Nevada or Coast source, except possibly the McCloud. A total development of 140 000 continuous horse-power is possible with the final tail-race at Elevation 1 000. It has recently been investigated by the California Débris and the Conservation Commissions for flood control. It will doubtless soon be used for hydro-electric development. It has no proponents, and no surveys or estimates as a supply for Greater San Francisco-Oakland have ever been made. The conduit line to Lake Chabot would be about 196 miles long.

By State legislation, this opportunity could be held indefinitely available for future use, at no expense except the economic waste resulting from preventing power development below 1 000 ft. elevation. The McCloud Project conduit line would be followed for about half the distance, the Yuba River conduit line would be common for most of the way.

Feather River.—The water-shed of the Feather River is characterized by steadiness (for California) and quantity of run-off, and by great storage possibilities. There is no doubt that it could furnish more water than Greater San Francisco-Oakland will ever need, in addition to all possible irrigation requirements. Unless the intake were at one of the large reservoirs in the head-waters, such as Indian Valley or Big Meadows, the purifying effect of storage would be offset by mixing with run-off from areas almost but not wholly uninhabited. If the supply is to be filtered, the lower Sacramento River, except in point of hardness, is more advantageous.

Yuba River.—There is little natural storage on the Yuba water-shed, except at the highest part of the South Yuba drainage area, therefore the run-off is rapid and irregular. By a system of natural and artificial reservoirs on the South and Middle Forks, a daily supply of 165 000 000 gal. of suitable water could be obtained at reasonable cost. This could be well adapted to a combination with water from some portion of the Feather River Basin, or other sources. The water is always more or less turbid. By making a diversion near the mouth, 400 000 000 gal. daily could be secured, but at so low an elevation

that pumping would be necessary and filtration advisable. Less than one-half the quantity could be spared by irrigation interests.

All these Sierra Nevada sources would be brought into Greater San Francisco-Oakland from the north, the conduits crossing Carquinez Straits. This, according to Mr. Wadsworth, would require 5 300 ft. of tunnel and 600 ft. of shafts, and would cost $1 150 000. Piercing the Mt. Diablo Range would be avoided. With the sources following Fig. 21, there would be no crossing of Carquinez Straits, but the Mt. Diablo Range would have to be passed.

American River.—The American River enters the Sacramento River at Sacramento. The Cosumnes, a tributary of the Mokelumne River, lies just to the south, and, owing to the topography, the upper and contiguous portions of the two water-sheds have usually been considered together in outlining power and water projects. A daily supply of about 230 000 000 gal. of excellent water can thus be obtained at reasonable cost, and diverted out of the valley without important injury to dependent irrigation interests. Approximately 62 000 net continuous horse-power could be developed incidentally.

Lake Tahoe.—Lake Tahoe is only mentioned because it seems naturally to suggest itself to many. It can supply daily about 275 000 000 gal. of excellent water, but all of it and much more is needed for the Nevada desert, where, logically and properly, it should be used.

Mokelumne River.—It is estimated that from the Mokelumne River a daily supply of from 130 000 000 to 150 000 000 gal. of satisfactory water could be secured. It is said that a storage of 250 000 acre-ft. is about the limit which can be developed on the water-shed, and that there are 200 000 acres of irrigable land dependent on the stream for water. In such case it is very doubtful whether it would not be against the best interests of society to divert out of the water-shed for any purpose more than half such quantity, or possibly any at all.

Stanislaus River.—Before getting even as far south as the Stanislaus River, the excess of water over satisfactory irrigation requirements has become very small, and here probably passes through zero and becomes minus. By taking into account the water due others by appropriations, it has been computed that about 60 000 000 gal. per day could be diverted out of the water-shed. The present water laws of California are little less than absurd, and it is most short-sighted to stop with them. Taking into account ultimate irrigation

needs, instead of rights, it is improbable that any water is properly available.

Tuolumne River.—The Tuolumne River is the source selected by Mr. Grunsky, City Engineer in 1901, and, because it contemplated transforming the famous Hetch Hetchy Valley into a reservoir, is generally known as the Hetch Hetchy supply. Mr. Grunsky proposed to bring in 60 000 000 gal. per day. In 1912 Mr. John R. Freeman increased the quantity to 400 000 000 gal. per day, or 618 sec-ft., and, among other results, was crystallizing opposition of local irrigation water users and securing by them conditions protecting many of their interests.

The Raker Bill, which Congress recently (December, 1913) passed, granted San Francisco the necessary reservoir, conduit, and other rights of way in, over, and through certain public lands, the Yosemite National Park, and the Stanislaus National Forest. Following the methods of the Conservation Policy, the grant requires:

1.—Filing customary maps, observance of National Park and Forest rules and of prior valid claims, etc.

2.—No sanitary regulations over the water-shed, other than as enumerated (those necessary in any event to protect campers in the Park from having polluted water to use).

3.—Recognition by the City of the prior right of the Modesto and Turlock Irrigation Districts (acreage not to exceed 300 000) to 2 350 sec-ft. of the natural daily flow at La Grange, whenever said districts can beneficially use it, and to 4 000 sec-ft. between April 15th and June 14th.

4.—The City to sell "such amounts of stored water as may be needed for the beneficial use of the said irrigation districts" at cost, with maximum and minimum quantities thereof which can be demanded during any calendar year. These quantities are to be fixed and occasionally revised by the Secretary of the Interior.

5.—The City not to divert out of San Joaquin Valley water for other than domestic and other municipal purposes.

6.—It defines "natural daily flow" as the quantity which "on any given day would flow in the Tuolumne River or its tributaries if said grantee had no storage or diversion works on the said Tuolumne water-shed."

7.—The immediate building of Hetch Hetchy Dam, 200 ft. high, and with a base "capable of supporting said dam when built to its greatest economical and safe height."

8.—The City to sell at cost electric power to said irrigation districts and municipalities within them.

9.—The installation by the City of 10 000 h.p. within 3 years of completing "any portion of the works adapted to the generation of electrical energy;" 20 000 h.p. within 10 years; 30 000 h.p. within 15 years; 60 000 h.p. within 20 years; and all additional possible power as soon thereafter as ordered by the Secretary of the Interior.

10.—The City to construct and forever maintain scenic roads and trails (estimated first cost $1 000 000).

11.—The City, after 5 years, to pay the Government $15 000 annually for 10 years, and thereafter $30 000 annually.

12.—The City to pay all costs of investigations or decisions required under the Act to be made by the Department of the Interior.

13.—The City to convey all its lands in the Yosemite National Park and the Stanislaus National Forest to the United States.

14.—The City to sell water at cost to the United States War Department in or near San Francisco.

The City of San Francisco formally accepted this grant with all these conditions on January 5th, 1914.

Under the Raker Bill, it is impossible to estimate closely either how much water San Francisco may divert or the cost of doing so. It is to finance the complete conservation of the water from the entire water-shed above La Grange and receive thereby the excess over local irrigation needs. The City assumes all the uncertainties. These are:

1.—Quantity of water needed for beneficial use by the 300 000 acres of land during critical years. Opinions vary from 600 000 to 1 050 000 acre-ft. per annum.

2.—Quantity of stored water needed for the beneficial use of the Irrigation Districts.

3.—Quantities the Secretary of the Interior will consider proper for the annual maximum and minimum quantities of stored water the City must furnish. This will be determined by the results of experience in the matter, and by public sentiment at that time, as reflected by the Secretary of the Interior, regarding the propriety of the City

having insisted on going for its water supply to a region where there is a deficiency instead of an excess.

4.—The available storage capacity in the water-shed. This has been usually taken at 800 000 acre-ft. The U. S. Geological Survey adopts 1 000 000 acre-ft. Mr. O'Shaughnessy, the present City Engineer, estimates it as 1 800 000 acre-ft.

5.—Quantity of run-off at the various reservoir sites and diversion points during critical years.

6.—Quantity of storage which the power projects and the irrigation districts will provide without cost to the City.

7.—Construction, depreciation, and operation cost of an undetermined quantity of storage.

The amount which, if invested at 4½% compound interest, in 1914, would construct the Freeman Hetch Hetchy project is estimated by Mr. Wadsworth as $38 901 000. This contemplates the initial installation of 160 000 000 gal. per day, 1914-20 (mean date 1917); 80 000 000 gal. per day additional, 1945-47; and 160 000 000 gal. per day additional (total 400 000 000 gal. per day), 1963-69.

These figures must be used cautiously, however, because:

1.—The City has revised the estimated cost of the first 160 000 000 gal. per day installation from Mr. Freeman's figure of $37 501 400 to about $64 000 000, and probably the figures on the other installations as well.

2.—No renewals of relatively short-lived constructions have been taken into account. There are about 100 miles of steel pipe, much of it under heads as high as 800 and 900 ft.

3.—The Freeman Hetch Hetchy project and the Raker Bill Hetch Hetchy project are very different things.

Net horse-power to the amount of 115 000 can be developed from the ultimate 400 000 000 gal. per day supply, at a cost for power-house and machinery installation of $6 000 000. A large part of this power must be disposed of at cost. For the remainder, the Railroad Commission will fix the rates, at least, practically.

So much for the cost to the City. There is also a cost to society, as any water diverted out of the shed will, within 25 years at most, be needed locally for irrigation (400 000 000 gal. per day (445 000

acre-ft. per annum) is probably enough to irrigate 200 000 acres of citrus lands or 130 000 acres of valley land). Assume the cost of putting such water on the land at $125 per acre, the value of land with water at $500, and without water at $50. The value of water, therefore, would be $325 per acre, 25 years hence. Taking 175 000 acres, the total is $56 875 000, and the present value of this (interest at 4½%) is $18 924 000.

This must also be used with caution, because:

1.—It is doubtful whether the City can obtain 400 000 000 gal. per day from this source to divert.

2.—It is not certain that all the water to be diverted is really needed for local lands. The exact limits of the irrigable area east of the 300 000 acres of the Turlock and Modesto Irrigation Districts have not been definitely determined. Neither is it known how much, if any, of this can be otherwise watered, from other streams, or by pumped wells. Where this may be practicable, the cost to society is the increase in irrigation costs capitalized.

3.—There are no reliable data to determine how many acres the diverted water would irrigate, nor the cost per acre of applying it.

4.—The time in the future, when the lands would otherwise be intensively used, may not be the 25 years taken. It is certain that relatively little of the land would be properly watered immediately, because the irrigationists have not now the financial ability. It would undoubtedly be done in less than 25 years, however.

Relative to this, it is well to remember that ultimately water will be taken from the Sacramento River near its mouth and carried south to help irrigate the insufficiently watered San Joaquin Valley. When this is done, the water will doubtless be taken along the western edge of the valley, where the rainfall and run-off is only a fraction of that on the west slopes of the Sierra Nevadas. It is certain that for at least 50 miles south of the Tuolumne, as far as Kings River, no Sacramento River water will be brought across the San Joaquin Valley to supplement the Sierra Nevada streams in their respective tributary valley areas.

In any event, the present value of the cost to society of the City's selecting this supply is tremendous. This fact, more or less clearly understood, is the cause of much very bitter feeling in the Turlock and Modesto Districts. This bitterness accentuates the un-

fortunate, but in some measure justifiable, feeling common in the City's back country that San Francisco is self-centered and selfish.

Approximate Estimates.—Although there are these several elements of uncertainty, some estimates of the quantity of water which can be secured and of the cost are necessary for comparative purposes, if for nothing else. Assume that 1 500 000 acre-ft. of storage can be provided in about 40 reservoirs very well distributed over the water-shed, and that such a storage system can be operated with 80% over-all efficiency,* including evaporation during the frequent 2 to 2½-year storage periods. From the hydrograph of the Tuolumne River at La Grange, Fig. 22, it would appear that, for the 2½ years ending November 1st, 1913, the total safe draft would have been about 1 200 000 acre-ft. per annum, with all reservoirs empty at the end of the period. During the 29 months the Tuolumne River would be dry below La Grange. Another critical period would have occurred in 1897-99. Judging from the precipitation record beginning in 1868 at La Grange—which is not a satisfactory indicator of conditions, but about the only one available—similar critical periods occurred, ending, respectively, November 1st, 1878 and 1888. In other words, such cycles seem to occur about every 10 years, covering 23% of the time. The average cost of 1 500 000 acre-ft. storage capacity would probably be at least $30 per acre-ft.,† or a total of $45 000 000. Other interests might be induced to provide half of this, or incidental power might be secured which would cause an equivalent result. Consequently, the cost to San Francisco of such a quantity of storage may be taken at between $22 000 000 and $23 000 000. As the water requirements of the irrigation districts are so much larger than those of the City, and as the ultimate use will develop so much more quickly, the mean date of construction would probably be about 1930. This would make the present value of storage cost $11 620 000.

The construction cost of the initial installation, 1914-20, exclusive of the Tuolumne water-shed storage, for 160 000 000 gal. per day, would probably be about $55 000 000, according to the City's

* To secure such an efficiency it would be necessary to sacrifice most of the great power possibilities of the water-shed, because of different water requirements. For this reason, the City in 1912 asked the U. S. Forestry Service to permit no water storage in the Tuolumne drainage area above La Grange which the City would not control.

† More probably the average cost for such a quantity of storage would be more than $40 per acre-ft. capacity.

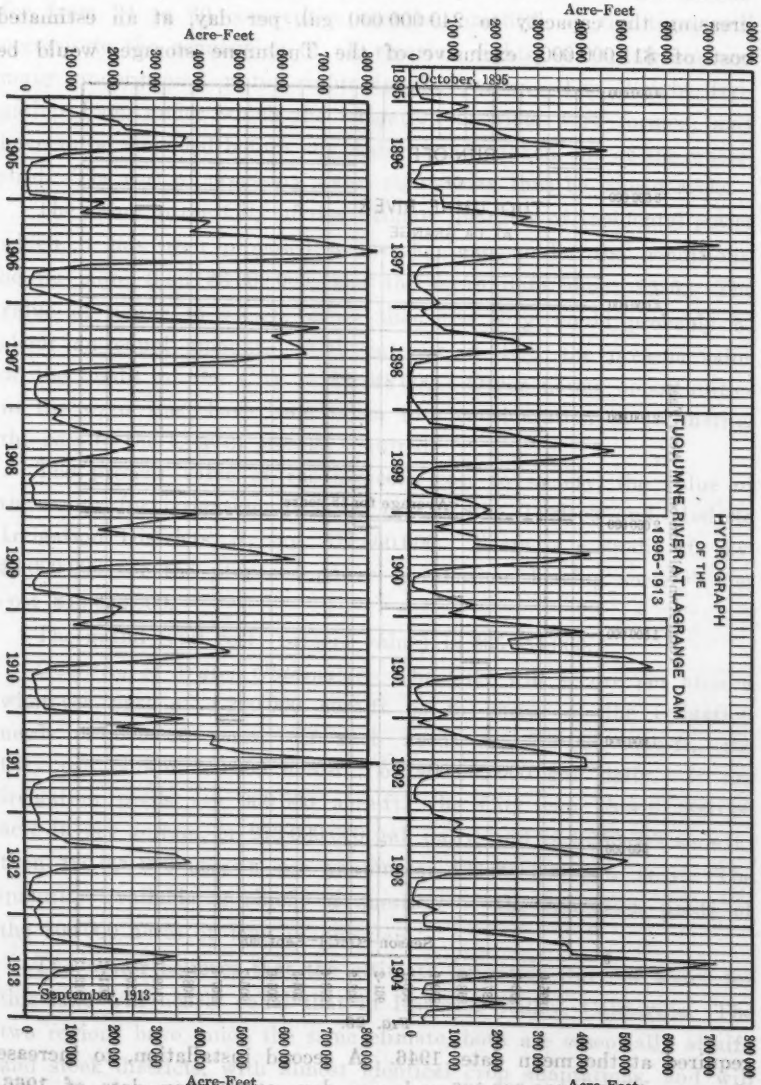

HYDROGRAPH
OF THE
TUOLUMNE RIVER AT LAGRANGE DAM
1895-1913
October, 1895
September, 1913
Acre-Feet
0
100 000
200 000
300 000
400 000
500 000
600 000
700 000
800 000
1895
1896
1897
1898
1899
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 22.

revised estimates. Using the consumption curves adopted by Mr. Wadsworth to secure comparative results, a second installation, increasing the capacity to 240 000 000 gal. per day, at an estimated cost of $11 000 000, exclusive of the Tuolumne storage, would be

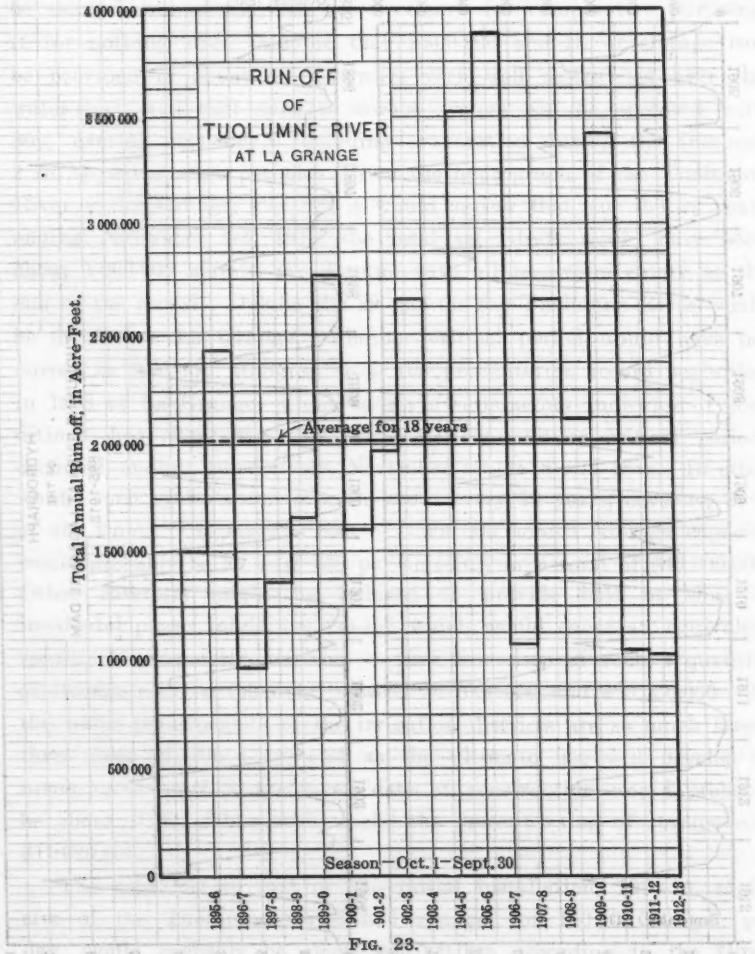


FIG. 23.

required at the mean date, 1946. A second installation, to increase the capacity to 400 000 000 gal. per day, at a mean date of 1966, will cost, exclusive of the Tuolumne storage, about $13 000 000. The present value of these expenditures is $51 650 000. Nothing is in-

cluded for renewals of short-lived materials, only initial costs. Plans which will make the Tuolumne River below La Grange dry for from 24 to 30 consecutive months, one-fifth of the time, will necessitate the acquisition of all riparian rights on the stream and many underground water rights in the region. The Wright Irrigation Law, under which the irrigation districts were formed, may have to be amended before any water can be taken out of the water-shed. The cost of necessary water rights may thus be considerable.

The value to the City of the ultimate quantity of incidental power which it has been proposed to develop can, under the conditions, be no more than $3 000 000, and the mean date about 1925. The value of power to society other than the City would probably be about an equal amount. With interest at 4½% the present value of the credit to the City is $1 848 000. Other power opportunities in the water-shed have been taken into consideration in estimating the cost to the City of storage required.

The present value of the project's cost to society, the value of the water for irrigation of lands east of the Turlock and Modesto Irrigation Districts, is, say, $17 500 000. There is a credit of, say, $3 000 000 for the proposed power installation, leaving society's net cost $15 652 000.

The grand total cost (present value) is $65 454 000.

Quantity of Water Obtainable.—The City will secure per annum whatever part of 1 200 000 acre-ft. is in excess of the irrigation needs of 300 000 acres. If such needs are 750 000 acre-ft., the City can obtain 450 000 acre-ft., or 400 000 000 gal. daily. If the irrigation needs are 900 000 acre-ft., the City can obtain 300 000 acre-ft. per annum, or 273 000 000 gal. daily, and if 1 050 000 acre-ft., then 150 000 000 acre-ft. per annum, or 136 000 000 gal. daily. The quantity available is chiefly a question of the irrigation needs of the 300 000 acres of land in question.

The writer believes that the ultimate irrigation duty of water for this land will nearly equal that in Imperial Valley, California. The two regions have much the same climate, both are essentially alfalfa and stock districts, with almost identical crop adaptations, and will doubtless have similar social conditions. The rainfall in the Turlock and Modesto Irrigation Districts averages about 9 in. and in

the Imperial Valley only 3 in., but in each case practically none occurs during the irrigation season. As an offset, the San Joaquin Valley lands are much more porous than those of Imperial Valley, which are the closest ever successfully irrigated on a large scale. All irrigation water in Imperial Valley is sold on a metered basis, and is used with care and economy. Indeed, practically no drainage system has ever yet been seriously suggested though the entire area is extraordinarily level. The irrigation canals are sealed with a deposition of fine Colorado River silt, so that the seepage loss of the distribution system is quite as low as could be expected from concrete-lined canals.* Imperial Water Co. No. 1 has 101 670 acres under actual cultivation in a compact body with the crops shown in Table 4.

TABLE 4.

Crop.	Number of acres.	Percentage.
Alfalfa	54 280	54.0
Alfalfa and barley	18 600	18.4
Barley	9 710	9.6
Corn	6 790	6.7
Cotton	6 150	6.0
Corn after barley	3 500	3.4
Melons	3 000	3.0
Miscellaneous	2 880	....
Alfalfa and corn	880	....
Vineyards	820	....

The net quantity of water actually delivered to the highest corner of each 160-acre tract of land during 1912 was 318 000 acre-ft., and in 1913, it was 325 000 acre-ft.

Safety.—As to relative safety, the presence of "active faults" in the Mt. Diablo Range, which will be crossed by the pressure tunnels, is a matter which many local engineers regard as very serious. Mr. Freeman holds the opposite opinion, and greatly prefers this construction to the Carquinez Straits crossing.

Quality of Water.—Water obtained from this source would be very soft (hardness about 20). All the contributory water-shed lies within the Yosemite National Park, and, therefore, will have no permanent population. On the other hand, with the improved transportation facilities to be made on the terms of the Raker Bill, the

*"Irrigation and River Control in the Colorado River Delta," *Transactions*, Am. Soc. C. E., Vol. LXXVI, pp. 1428-1431.

number of tourists, summer visitors, and campers on the water-shed will soon become a matter of importance. Taking this into consideration, together with the constantly rising standards in the quality of water supplies, Mr. Allen Hazen and Dr. Rupert Blue, Chief Surgeon, U. S. Navy Department, at the hearing before Secretary Fisher, in November, 1912, agreed in the opinion that complete filtration would be required in about 50 years, possibly sooner. Except for being softer, it would, under existing conditions, be almost exactly on a par with the water now delivered from Crystal Springs Reservoir. Judging from small reservoirs in the mountain regions, there is a possibility that greater trouble would be caused by minute organisms, such as algæ.

Possibility of Enlarging Supply.—If all the Bay communities pool their water supplies, distant supplies will probably not be required before 1945, and by 1990—taking the upper limit of 400 000 000 gal. per day as the quantity which may be obtained from this source—the combined sources will become inadequate within a period of 45 years from their introduction. Long before the latter date, it will be out of the question to secure any more supplies from that general region, and additional water will be obtainable only from the Sacramento River or some of its tributaries, that is, some of the northern sources, the construction of any one of which would be entirely independent of the Tuolumne works.

Combination of Sources.—Obviously, numerous combinations of the various Southern Sierra sources can be made, as well as of those of the Northern Sierras. Some of these may offer considerable advantages.

Conclusions.

Summarizing the salient features of the needlessly complexed political, sociological, and engineering problem, it appears to the writer that the citizens of San Francisco should take stock, and determine:

A.—Whether the water problem of San Francisco is primarily one for San Francisco alone or for Greater San Francisco-Oakland.

B.—Whether San Francisco should attempt to shoulder the financing and responsibility of an entire metropolitan district supply.

C.—Whether the present water system and sources supplying San Francisco proper, when owned actually by the City and properly improved and extended, are not satisfactory and adequate for the needs of the main City for the next two or three generations.

D.—Whether the other portions of Greater San Francisco-Oakland, and particularly the cities of Oakland, Berkeley, and Alameda, are not far more concerned than San Francisco in distant water supplies.

E.—Whether, on sentiment, San Francisco can afford to invest, in a new and distant water plant, sums so vast that many other requirements more vital to the interests of the City must be sacrificed or skimped.

F.—Whether the Hetch Hetchy scheme in its final form is a sound and practical one, from the engineering standpoint, and whether engineers of practical western experience, who are best qualified to pass on its peculiarly western problems, generally endorse or generally question it.

G.—Whether the Raker Act does not strip from the Hetch Hetchy project most of its advantages and assets, leaving the City in the position of "holding the bag" for others.

H.—Whether, all things considered, the only wise line of policy for San Francisco to follow is not:

First, to consummate the acquisition of the present water system, with all its appurtenances;

Thereafter, to take the water situation out of politics by having water matters handled by a Water Board of very wide powers, and entirely independent of all other City authorities;

Thereafter, to improve, extend, and amplify that system to meet fully all the City's needs for 70 or 80 years to come;

And, thereafter, to weigh carefully the relative advantages of entering into the vast Hetch Hetchy scheme or taking up some of the more important sociological problems, with a view to the more rapid and substantial development of the City itself, and let the other Bay cities work out their own water destiny for themselves.

Also, that the citizens of the East Bay cities should consider and decide, in addition to some of these same things, the following:

A.—Whether they should not acquire the existing water-works on the mainland, and at once improve, extend, and amplify them fully.

B.—Whether, as these cities are really in competition with Los Angeles rather than San Francisco, distant sources necessitating rates exceeding 15 cents per 1 000 gal. can be afforded.

ACKNOWLEDGMENTS.

The writer is indebted to many engineers and others for valuable suggestions and other assistance in preparing this paper. It is desired to extend grateful acknowledgment and appreciation to all of them. Especial thanks are due J. H. Dockweiler and C. E. Grunsky, Members, Am. Soc. C. E., particularly for the use of private reports and data.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA.

BY E. L. SAYERS AND A. C. POLK, MEMBERS, AM. SOC. C. E.

TO BE PRESENTED NOVEMBER 4TH, 1914.

SYNOPSIS.

The object of this paper is to describe the design and construction of a low-head hydro-electric plant recently completed in Alabama, and thereby bring out a discussion, if possible, on the latest ideas in the practice of developing water-power under low heads, the latest features in hydraulic turbine design, modern methods of transmission, and the construction of such plants.

The Alabama Traction, Light, and Power Company was organized in 1912 for the purpose of developing the water-powers of Alabama. The Company acquired several public service companies operating steam generating plants, street railways, gas plants, etc., and several companies owning undeveloped water-power sites. Among the latter was the Alabama Power Company, which owns a site known as "Lock 12" on the Coosa River at a central point in the State.

This plant will form the hub of an extensive system of power

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

plants capable of developing 24-hour, 80% efficiency power in excess of 500 000 h.p.

The foundations of the power-house are built transversely to the axis of the river and form an integral part of the dam. The length of spillway, power-house, and abutments, over all, is 1 530 ft. 5½ in. The power-house substructure is 303 ft. 4 in. long, 136 ft. 5 in. wide, and contains provision for six units. Four units constitute the present installation, and the superstructure is built with a temporary wall at the river end so that it can be extended to cover all six.

The spillway forming the principal part of the dam has a net crest length of 780 ft., divided into twenty-six sections, each 30 ft. long, separated by concrete piers, 6 ft. long on the axis of the dam. On the crest of each section of spillway there is placed between piers a gate, 30 ft. long and 14 ft. high, of a modified Stoney type. The height of the dam from the average elevation of the original river bed to the elevation of the flow line of the reservoir is 77 ft.

In all, there are approximately 190 000 cu. yd. of concrete in the power-house foundations and the dam. The first concrete was placed on April 12th, 1913; it was all placed by February 26th, 1914; and commercial power was turned out for the first time on April 12th, 1914, just one year, to the day, from the date of placing the first concrete.

The installation is made up of four vertical, single-runner turbines, guaranteed to deliver to the shaft, under 68 ft. head, at 100 rev. per min., 17 500 h.p. One feature of these units, as described in the paper, is that they are entirely self-contained, that is, they can be built up on an assembly floor from the foundation ring to the top of the direct-connected exciter above the generator, without requiring any lateral support.

Another feature of the development, as described in the paper, is the use of an improved volute casing, through which the water passes from the penstocks to the wheels. These scroll casings were complicated in form, and were moulded directly in the concrete of the power-house foundations, without the use of a steel lining.

After giving a brief description of the general system of the Alabama Traction, Light, and Power Company and of the power market to be served, the paper describes the Coosa River and the cli-

matic conditions under which the work was done, so that a better idea can be had by those not familiar with the locality.

The machinery is fully described, greater detail being given in the case of the turbines than for the electrical equipment, discussions of which are constantly appearing in papers of electrical societies and in the electrical technical press.

In connection with the construction of the plant, the materials used and their sources, the general scheme of handling the work and its prosecution, are dealt with in detail. The selection of plant, its layout, and the construction of camps are described, and improvements which experience showed could have been used, are noted.

The concrete forms used in the power-house foundations were necessarily complicated, and the details of their design and construction are shown. The various methods used in placing concrete and the success attending their use are described.

At the end of the paper, the design and construction of transmission lines and sub-stations are dealt with briefly.

Introductory.

In the last 10 years the progress made in methods of generating electric power and transmitting it over long distances to markets has been very marked, and although more or less complete descriptions of hydro-electric developments have been appearing in various technical papers, there has been no extended discussion of modern methods before this Society. This fact appears the more strange because at the present time a large amount of work is being done in developing the water-powers of the United States, and this branch of the Profession is fast becoming one of the most important.

No extraordinary features are claimed for the work described. The plant develops power under the moderate head of 68 ft., the installation of 70 000 h.p. is not remarkable, and no great difficulties in construction were met. It is believed, however, that it represents the latest improvements in equipment, and that a full discussion of this development and others of similar proportions will be of practical value to the Profession by bringing before it the latest practice in developing water-power under low heads, the latest features in hydraulic turbine design, modern methods of transmission

of high-potential current, and methods of construction which have been tried and found successful.

General System of the Alabama Traction, Light, and Power Company.

The Alabama Traction, Light, and Power Company, organized in 1912, acquired the properties, rights, and franchises of several public service companies in Alabama, and of several companies owning water-power sites in the State. Among the latter were the Alabama Power Company, with several undeveloped sites on the Coosa River; the Alabama Interstate Power Company, with a site on the Tallapoosa River; the Little River Power Company, with a site on Little River; and the Muscle Shoals Hydro-Electric Power Company, owning three sites on the Tennessee River. These sites represent a possible development in excess of 500 000 h.p. continuous 24-hour power.

The developments proposed by the Company on the Coosa are at the sites selected by the United States Government engineers for dams and locks for the improvement of the river. Fig. 1 is a profile of the Coosa River covering that portion affected by the proposed developments. The dam farthest south, at which power could be developed, is at a point 7½ miles above Wetumpka. This site, known as Lock 18, has an effective head of 63 ft., and the dam would raise the water to Elevation 254 (U. S. Army datum) from Elevation 190. At Lock 15, the next site, 19½ miles above Wetumpka, the water would be raised from Elevation 254 to Elevation 294, giving an effective head of about 39 ft. At a point about 34 miles above Wetumpka, the dam at Lock 14 would raise the water to Elevation 352, giving an effective head of about 57 ft. Lock 12 is 40 miles above Wetumpka, and here the water level is raised to Elevation 420, making the effective head of 67 ft. These sites are all shown on the general map, Fig. 2.

The proposed development on the Tallapoosa River is at Cherokee Bluffs, about 30 miles northeast of Montgomery. Here a dam 180 ft. high would back the water up 30 miles in the river, making one of the largest storage reservoirs in the country, the area being 34 000 acres.

The Little River development is in the northeastern part of the State (see Fig. 2), about 33 miles northeast of Gadsden. The devel-

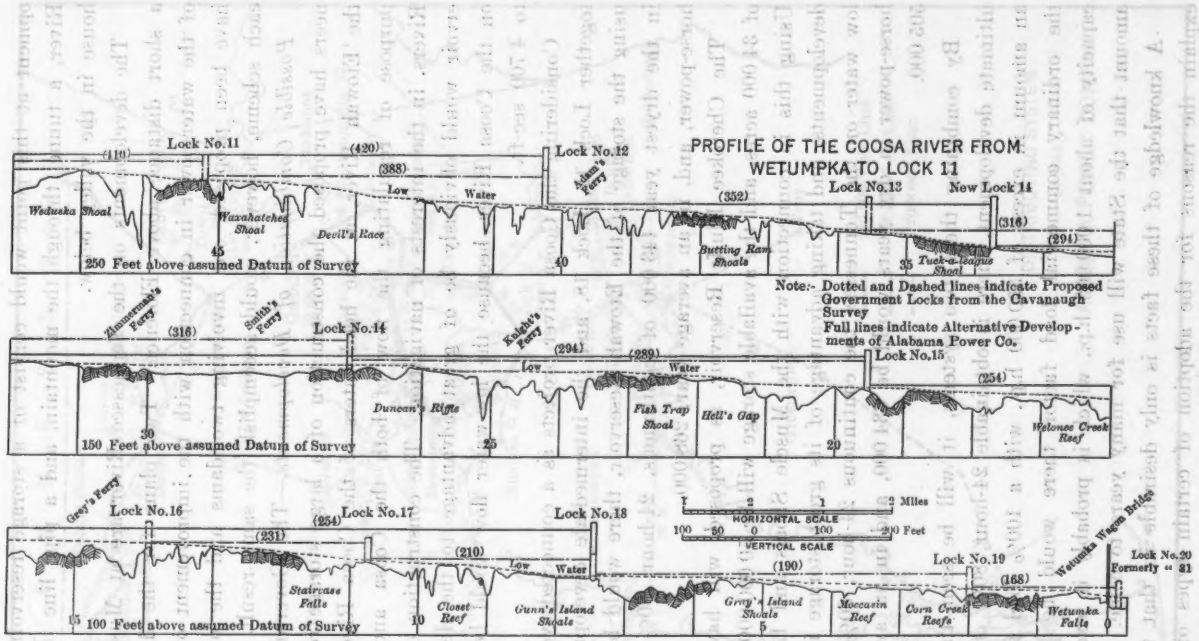

PROFILE OF THE COOSA RIVER FROM WETUMPKA TO LOCK 11
Lock No.11
Lock No.12
Lock No.13
New Lock 14
Lock No.14
Lock No.15
Lock No.16
Lock No.17
Lock No.18
Lock No.19
Lock No.20
Formerly " 21
Adam's Ferry
Zimmerman's Ferry
Smith's Ferry
Knight's Ferry
Grey's Ferry
Wetumka Wagon Bridge
Wetumka Shoal
Waxahatchee Shoal
Devil's Race
Butting Ram Shoals
Tuck-a-league Shoal
Duncan's Riffle
Fish Trap Shoal
Hell's Gap
Welonee Creek Reef
Staircase Falls
Closet Reef
Gunn's Island Shoals
Grey's Island Shoals
Moccasin Reef
Corn Creek Reefs
Wetumka Falls
Low Water
250 Feet above assumed Datum of Survey
150 Feet above assumed Datum of Survey
100 Feet above assumed Datum of Survey
Note:- Dotted and Dashed lines indicate Proposed Government Locks from the Cavanaugh Survey
Full lines indicate Alternative Developments of Alabama Power Co.
HORIZONTAL SCALE
VERTICAL SCALE
Miles
Feet

FIG. 1.

opment at this point would consist of a storage reservoir on Little River, a tunnel through the mountain, and a pipe line to a power-house in the valley below.

The developments on the Tennessee River are at Muscle Shoals, a short distance above Florence. Two plans for the development of the water-power in connection with the improvement of the river have been proposed, one involving two dams and the other three, each scheme, however, would accomplish the same result.

Possible Combination of Developments.—The Government engineers have proposed the construction of a large storage reservoir on the Etowah River, at the head-waters of the Coosa River, for the purpose of regulating the flow of both the Coosa and Alabama Rivers, in the interests of navigation. The construction of this reservoir would obviously be of great advantage to the developments on the Coosa River, because the low-water flow would be increased to 4 700 sec-ft.

Considering the Coosa River projects as a connected system, tying together Lock 12, Lock 18, and the intermediate developments, and using the storage of the Etowah Reservoir, there would be produced in the dryest year 143 000 of continuous, 24-hour, 80% efficiency, horse-power, and, in an average year, 268 000.

The Cherokee Bluffs Reservoir, as proposed, will have an area of 34 000 acres, and the available storage will be 50 000 000 000 cu. ft. Using this in connection with the Muscle Shoals and Little River developments, and taking advantage of its great storage in times of low water on the Tennessee, the continuous 24-hour, 80% efficiency, horse-power in dry years would be 334 000, and in an average year 505 000.

By combining these two systems, it will be seen that, in the ultimate development, there is obtainable 24-hour, 80% efficiency, to an amount in excess of 550 000 h.p., with a 100% load factor. On the ordinary commercial load factor there would be required a capacity of about 1 000 000 h.p., which is probably in excess of the amount that the State will use for many years to come.

A knowledge of these facts is only desirable in that they partly explain the reasons for the adoption of certain types of construction as being part of an extensive proposed system.

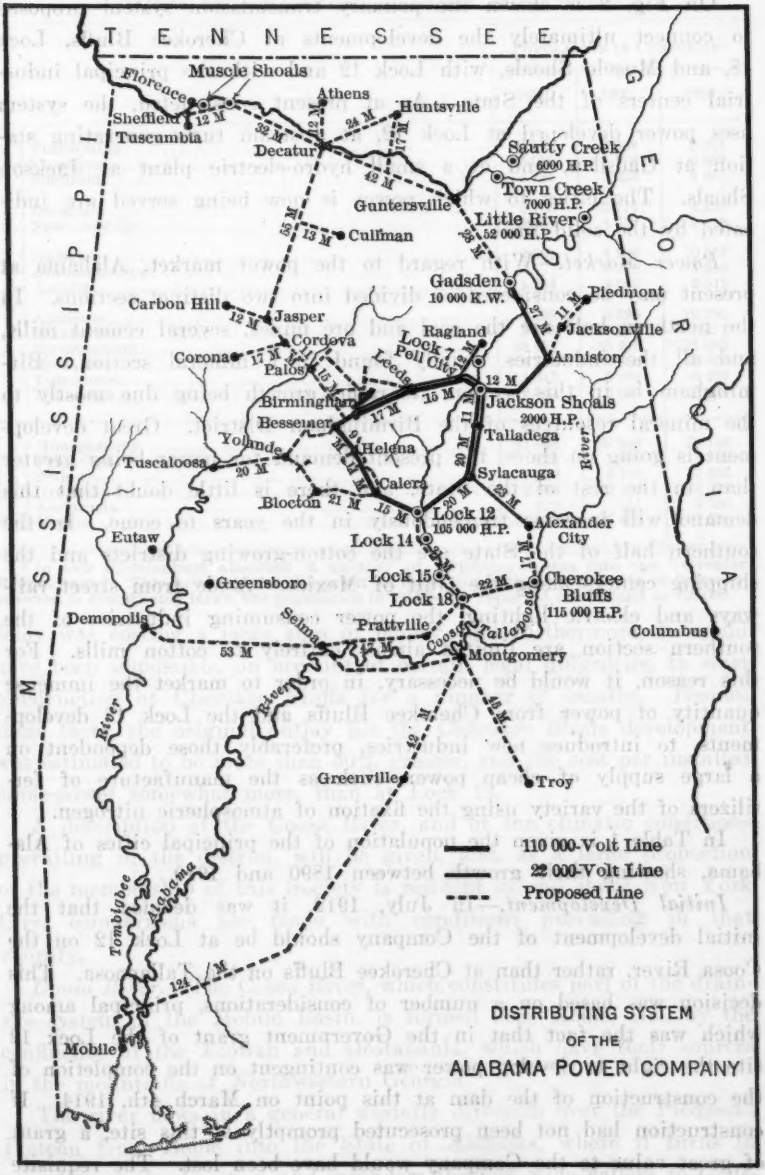

TENNESSEE
Muscle Shoals
Florence
Athens
Huntsville
Sheffield
Tuscumbia
Decatur
Scottsboro
Scutty Creek
6000 H.P.
Town Creek
7000 H.P.
Guntersville
Little River
52 000 H.P.
Cullman
Gadsden
10 000 K.W.
Piedmont
Carbon Hill
Jasper
Cordova
Ragland
Lock 7
Pell City
Jacksonville
Corona
Palos
Leeds
Anniston
Birmingham
Jackson Shoals
2000 H.P.
Bessemer
Talladega
Helena
Tuscaloosa
Sylacauga
Calera
Blocton
Lock 12
105 000 H.P.
Alexander City
Eutaw
Lock 14
Greensboro
Lock 15
Lock 18
Cherokee Bluffs
115 000 H.P.
Demopolis
Prattville
Columbus
Montgomery
Greenville
Troy
Mobile
110 000-Volt Line
22 000-Volt Line
Proposed Line
DISTRIBUTING SYSTEM
OF THE
ALABAMA POWER COMPANY

FIG. 2.

On Fig. 2 is shown the primary transmission system proposed to connect ultimately the developments at Cherokee Bluffs, Lock 18, and Muscle Shoals, with Lock 12 and with the principal industrial centers of the State. As at present constructed, the system uses power developed at Lock 12, at a steam turbo-generating station at Gadsden, and at a small hydro-electric plant at Jackson Shoals. The cities to which power is now being served are indicated by the solid lines.

Power Market.—With regard to the power market, Alabama at present may be considered as divided into two distinct sections. In the northern half are the coal and ore mines, several cement mills, and all the industries usually found in a mineral section. Birmingham is in this section, its rapid growth being due mostly to the mineral resources of the Birmingham District. Great development is going on there, the present demand for power being greater than in the rest of the State, and there is little doubt that this demand will increase tremendously in the years to come. In the southern half of the State are the cotton-growing districts and the shipping centers along the Gulf of Mexico. Aside from street railways and electric lighting, the power consuming industries of the southern section are limited almost entirely to cotton mills. For this reason, it would be necessary, in order to market the immense quantity of power from Cherokee Bluffs and the Lock 18 developments, to introduce new industries, preferably those dependent on a large supply of cheap power, such as the manufacture of fertilizers of the variety using the fixation of atmospheric nitrogen.

In Table 1 is given the population of the principal cities of Alabama, showing their growth between 1890 and 1910.

Initial Development.—In July, 1912, it was decided that the initial development of the Company should be at Lock 12 on the Coosa River, rather than at Cherokee Bluffs on the Tallapoosa. This decision was based on a number of considerations, principal among which was the fact that in the Government grant of the Lock 12 site the right to develop power was contingent on the completion of the construction of the dam at this point on March 4th, 1914. If construction had not been prosecuted promptly at this site, a grant of great value to the Company would have been lost. The requisite funds for the initial development had been raised, and each day's

TABLE 1.—POPULATION OF CITIES IN ALABAMA.

	Cities.	Population in Census of:		
		1890.	1900.	1910.
Northern Alabama.	Tuscumbia	2 491	2 348	3 324
	Florence	6 012	6 478	6 689
	Sheffield	2 731	3 333	4 865
	Decatur	2 765	3 114	4 228
	New Decatur	3 565	4 437	6 118
	Gadsden	2 901	4 282	10 557
	Alabama City	.....	2 276	4 313
	Attalla	1 254	1 692	2 513
	Anniston	9 998	9 695	12 794
	Huntsville	7 995	8 068	7 611
	Birmingham	26 178	38 415	*132 685
	Bessemer	4 544	6 358	10 864
	Tuscaloosa	4 215	5 094	8 407
	Talladega	2 063	5 056	5 854
Southern Alabama.	Montgomery	21 883	30 346	38 136
	Prattville	724	1 929	2 222
	Selma	7 622	8 703	13 649
	Greenville	2 806	3 162	3 377
	Andalusia	270	551	2 480
	Mobile	31 676	38 469	51 521

*In 1909 Birmingham absorbed a number of adjoining towns into the "Greater Birmingham." This accounts for the great increase in population; the rate of increase, however, is remarkably large, the population in 1914 being reported unofficially as 166 000.

delay was costing a large sum of money, and furthermore, it would have been impossible, on account of certain legal difficulties, to start construction at Cherokee Bluffs for a number of months. Besides these facts, the original outlay for the Cherokee Bluffs development was estimated to be more than 50% greater, and the cost per installed horse-power somewhat more, than at Lock 12.

A description of the Coosa River, and of the climatic conditions prevailing in the district, will be given, and, as a large proportion of the membership of this Society is resident in and about New York City, comparisons are made with conditions prevailing in that vicinity.

Coosa River.—The Coosa River, which constitutes part of the drainage system of the Mobile Basin, is formed near Rome, Ga., by the confluence of the Etowah and Oostanaula, which have their sources in the mountains of Northwestern Georgia.

The river flows in a general westerly direction over the Piedmont Plateau from Rome into the State of Alabama, where it turns in a southwesterly direction and meanders through wide valleys to Gads-

den. Here, again, the river changes its general trend, turning more toward the south and flowing in a more direct line, between mountains of limestone formations. A short distance below the crossing of the Louisville and Nashville Railroad about opposite Sylacauga, the river passes out of the limestone formations into the slates and schists, and the bed of the stream becomes hard and very rough. From this point the river flows in a southeasterly direction over a succession of shoals and riffles to Wetumpka, where it joins the Tallapoosa River to form the Alabama. In this last section, where the fall in the river is more abrupt, a large quantity of power can be developed economically.

The first project for improving the river for navigation was proposed in 1875, and some work has been done in the general scheme of making the river navigable for light-draft boats. The system, when completed, will be a waterway, almost 900 miles long, from the head-waters of the Coosa to the Gulf of Mexico. Prior to 1912, however, no improvements had been made in the rough section of the river between Gadsden and Wetumpka.

In its upper reaches the Coosa flows through clay-covered territory where it accumulates from its feeders, in all except extreme low-water seasons, a quantity of silt, making it appear very muddy. Tests, however, have shown that this extremely muddy appearance is due in great part to material which has gone into solution in the water, as at its worst it carries in suspension not more than 1 part of silt in 6 000. It is not anticipated, therefore, that there will be any considerable difficulty from silting of reservoirs on this river.

The source of the river is in a region which has a heavier annual rainfall than any section of the United States, with the exception of some parts of the State of Washington. The surface rocks are mostly limestones, which, of course, are only moderately porous; but they are broken up with large fissures, and the solvent action of the rainfall has converted many of these fissures into subterranean channels, as shown by the number of large springs which abound in this territory. To a certain extent, this condition, has a regulating effect on the watercourses emptying into the Coosa, yet in the winter and early spring of the normal year the rains are intense, and the river is subject to floods from December until April, with occasional flashy floods before and after that period, the month of greatest flow being March. The regulating effect of these springs and the fact that in

the average year the rainfall is distributed somewhat uniformly over the months are responsible for the fact that the low-water flow of the Coosa is ordinarily nearly three times as great as that of the Hudson at Albany, and is equal to the low-water discharge of the Mississippi at St. Paul. Table 2 gives the discharge record of the river from 1897 to 1914, inclusive.

The greatest flood known in the river occurred in the spring of 1886, before gauge readings had been commenced. At Lock 12 this flood has been variously estimated, by engineers who have studied and reported on the Coosa River projects, to have been from 95 000 to 170 000 sec-ft. Extensive studies made during 1913 have indicated that the probable flood intensity was approximately 135 000 sec-ft. The average spring flood in the last 14 years, during which time observations have been taken continuously has amounted to 80 000 sec-ft.

The area of the water-shed above Lock 12 is 9 087 sq. miles. Using the estimated maximum flood of 1886, 135 000 sec-ft., this would give a maximum discharge of 14.8 sec-ft. per sq. mile of water-shed, which is one of the smallest in the country. This is explained principally by the fact that the water-shed is very large, and the average intensity of rainfall during a storm is very small. To illustrate this, a few examples of flood discharges are given in Table 3.

The lowest flow known to have occurred was in 1904, at which time observations were taken at Riverside, 72 miles above Lock 12. A comparison of the gauges and rating curves at these two points indicates that the minimum flow was 1 700 sec-ft. at Lock 12. The average minimum flow since observations have been taken amounts to 5 000 sec-ft. With the regulation effected by the construction of the dam at Lock 12, the minimum flow of 1904 would have amounted to 2 400 sec-ft.

Climatic Conditions.—Few people in the North realize that Birmingham, Ala., has had a greater range of temperature, both at the upper and lower limits, than New York City. The records of the United States Weather Bureau show that the maximum temperature of New York City is 100° and that the minimum is — 6°, and that the temperature has ranged between 104 and — 10° at Birmingham.

In summer the heat is intense, but of an entirely different character from that of New York City. For long periods the temperature during the day ranges between 90° and 100°, but heat prostrations

TABLE 2.—DISCHARGE OF THE COOSA RIVER

Year.	JANUARY.		FEBRUARY.		MARCH.		APRIL.		MAY.		JUNE.	
	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
1897 to 1913, inclusive.	19 081	94 150	29 478	106 200	35 447	125 700	25 771	112 500	14 184	71 500	13 107	85 500
1897	10 800	30 800	25 100	42 250	55 000	91 000	23 500	66 500	9 150	13 300	6 500	7 250
1898	15 500	55 000	8 400	22 800	8 600	20 200	25 100	56 000	7 200	13 000	5 800	8 200
1899	14 000	22 300	46 750	83 250	61 500	107 250	35 000	67 750	10 300	17 750	7 250	10 300
1900	17 200	49 750	32 500	91 000	38 300	72 000	43 700	112 500	10 800	19 250	30 500	85 500
1901	36 600	94 150	29 650	65 300	28 000	95 700	45 300	84 800	21 150	64 550	15 800	36 900
1902	32 900	94 150	34 600	85 500	53 000	106 500	28 250	92 500	8 750	11 000	6 750	7 400
1903	14 200	26 300	62 150	106 200	57 600	89 800	37 250	79 000	13 350	40 000	18 500	50 200
1904	7 750	18 250	11 000	23 200	14 250	27 750	10 000	18 700	6 400	8 400	6 300	9 650
1905	18 300	73 100	39 000	87 100	14 700	26 800	9 800	11 750	18 500	58 750	8 750	13 000
1906	30 300	64 600	11 200	20 600	50 500	125 700	19 000	56 000	9 300	14 150	13 150	46 000
1907	20 200	70 000	26 000	61 500	26 150	80 800	13 700	24 100	20 650	57 600	14 000	40 750
1908	23 700	53 000	43 200	83 500	27 000	74 650	20 500	37 250	13 300	18 700	8 500	12 200
1909	19 250	46 750	43 750	81 750	70 000	124 000	25 150	63 800	28 300	68 500	33 750	77 800
1910	10 800	21 750	15 850	49 100	15 000	40 800	8 500	17 750	24 000	71 500	15 900	28 750
1911	17 200	67 000	14 150	29 400	9 750	16 800	33 200	84 750	8 750	13 200	6 250	7 250
1912	16 400	40 024	28 350	52 380	37 800	59 620	34 380	61 770	17 050	39 172	12 020	29 800
1913	27 258	89 000	40 968	64 500	60 845	82 560	22 802	55 700	9 772	17 260	8 237	12 280

are practically unknown. The evenings are cool, and the breezes which blow during the night are refreshing after the heat of the day. In its effect on the speed of the work, and consequently its cost, this heat is to be reckoned with, because labor automatically adapts itself to the climate.

TABLE 3.—FLOOD DISCHARGE OF SEVERAL RIVERS.

Stream.	At	Water-shed, in square miles.	Cubic feet per second per square mile.
Six-Mile Creek	Ithaca, N. Y.	47.5	170
Gallinas River	Las Vegas, N. Mex.	90.0	129
Tohickon Creek	Mt. Pleasant, Pa	102.0	112
Nashua River	Massachusetts	109.0	104
Mora River	La Cueva, N. Mex.	159.0	140
Croton River	Croton Dam, N. Y.	339.0	74
Broad River	Carlton, Ga.	762.0	38
Raritan River	Bound Brook, N. J.	879.0	59
Mohawk River	Little Falls, N. Y.	1 306.0	22
Tallapoosa River	Milstead, Ala.	3 840.0	18
Hudson River	Mechanicsville, N. Y.	4 500.0	15
Kanawha River	Charlestown, W. Va.	8 900.0	13
Coosa River	Lock 12, Ala.	9 087.0	15
Tennessee River	Chattanooga, Tenn	21 418.0	21
Susquehanna River	Harrisburg, Pa.	24 030.0	19
Mississippi River	St. Paul, Minn.	36 085.0	20
Kansas River	Lecompton, Kans.	58 550.0	4

The significance of the low limit of temperature reached at Birmingham is not as serious as the records would seem to indicate. Periods of low temperature are very rare, and have never been of

AT LOCK 12, FROM 1897 TO 1913, INCLUSIVE.

JULY.		AUGUST.		SEPTEMBER.		OCTOBER.		NOVEMBER.		DECEMBER.	
Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
11 991	64 500	10 312	72 800	8 128	62 250	8 895	95 000	7 944	72 400	14 130	96 400
9 250	35 250	6 300	8 400	5 200	6 100	5 050	6 250	5 000	5 550	8 750	17 500
6 800	13 400	12 000	26 800	16 300	62 250	24 000	95 000	10 300	28 700	9 400	13 400
7 800	18 750	6 400	13 200	6 200	9 000	5 450	5 800	5 800	9 650	13 750	36 900
17 500	62 250	7 600	16 250	9 000	31 800	9 500	31 800	10 300	43 800	19 000	30 600
9 000	13 800	27 500	72 800	13 000	31 800	8 150	13 350	6 700	7 150	25 250	96 400
6 250	9 200	6 100	7 900	6 500	13 300	7 000	13 300	6 600	13 000	16 300	33 800
9 950	26 250	8 000	17 500	5 500	7 000	5 600	6 000	5 500	6 700	5 400	5 800
6 000	8 400	8 600	24 250	5 100	5 600	4 500	4 800	4 950	5 100	7 650	18 750
10 700	24 600	9 300	18 700	6 000	7 900	6 600	10 000	5 750	5 800	28 750	56 800
28 700	72 400	14 500	22 900	13 550	19 200	27 200	69 250	23 200	72 400	15 900	46 000
8 450	11 750	7 500	9 650	8 800	29 300	6 700	13 300	12 200	44 500	46 800	38 000
7 500	15 000	7 800	12 150	6 300	13 750	5 600	9 500	5 700	6 300	20 000	63 750
13 750	28 750	15 000	50 500	7 150	11 400	6 750	18 700	5 750	5 800	10 000	20 700
27 000	64 500	8 800	17 750	7 500	15 400	6 000	7 190	5 500	5 600	7 790	24 500
8 550	15 900	9 300	19 200	5 300	5 600	6 700	19 200	8 000	19 650	16 250	56 000
14 650	29 250	10 290	32 000	8 657	16 100	7 524	13 300	5 860	7 320	11 186	19 600
6 752	12 700	6 760	10 500	4 403	17 400	5 635	21 600	3 315	4 000	6 774	14 100

long duration. Ice is never formed to any thickness in ponds or rivers, and the mid-day sun almost invariably causes the temperature of the coldest days to rise above freezing.

It is not necessary, therefore, to consider the effect of an ice thrust in the design of a dam in this climate, and, for the same reason, no precaution need be taken in the design of forebays, for the care and handling of ice at penstock screens. Needless to say, no difficulties are encountered with frazil.

To make clearer the rainfall conditions, Table 4 shows a comparison between the records of normal monthly precipitation at New York City and at Birmingham. From this it will be seen that the rainfall in New York City is more evenly distributed throughout the year than in Birmingham, and that, from December to March, inclusive, the Birmingham rainfall is far in excess of the mean monthly. In July and August, the precipitation at each place is about equal and is the result of heavy showers of short duration.

In the normal year, construction work in Alabama is hampered to a large extent in winter by rains of several days' duration. At such times it is impossible to carry on work, because laborers, even if provided with suitable clothing, will not turn out.

Work Done to Date.—In April, 1912, E. A. Yates, Assoc. M. Am. Soc. C. E., was appointed Chief Engineer of the Company, and an en-

gineering and construction force was organized. A party was put in the field at Cherokee Bluffs for the purpose of surveying the flooded area (34 000 acres) of that reservoir. A short time later an additional party was placed in this reservoir, and a large party was put to work on the surveys for the Coosa River reservoirs. The surveys for the Muscle Shoals reservoirs were made in 1913.

TABLE 4.—COMPARISON OF RECORDS OF NORMAL MONTHLY PRECIPITATION AT NEW YORK CITY AND AT BIRMINGHAM, ALA.

Month.	New York City.	Birmingham, Ala.
January	3.79	5.32
February	3.74	4.75
March	4.10	5.76
April	3.30	3.67
May	3.18	3.09
June	3.26	3.88
July	4.54	4.70
August	4.53	4.48
September	3.59	3.50
October	3.71	2.34
November	3.44	3.39
December	3.45	4.60
Year	44.63	49.48
Mean Monthly	3.72	4.12

At the time the decision had been reached that the initial development should be at Lock 12, no plans of the development had been drawn up, with the exception of a general plan and elevation of the proposed structure. Only 1½ years remained for the construction of the dam within the time stipulated in the Government grant, and, therefore, it was impossible to draw up complete plans before proceeding with the construction. It was desirable to let a contract for the construction of the power-house foundations and dam to a contractor having plant and equipment ready to be moved to the work without delay. Such a contract was let, for the construction of the power-house foundations and dam, on a cost-plus-fixed-fee basis, and the direction of the work, approval of selection of plant, and methods of attack were reserved to the Chief Engineer of the Company. It was decided that the construction of the superstructure and the equipment of the power-house should be done by the Company's own force. As soon as this contract was signed, the necessary preparations for starting construction were inaugurated.

An old lumber railroad (shown on Fig. 3), extended through the hills from a point where it had physical connection with the Louisville and Nashville Railroad, about 5 miles south of Calera, to a point about 5 miles from the dam site. It was known as the Clear Creek Railroad. It was not ideal for construction purposes, because, in its length of 18 miles, it had forty-two timber trestles, one switch-back, and many sharp curves, the sharpest being 35° on a 2.5% grade. The shortest, alternate line, however, would have been at least 14 miles long, and would have involved construction from beginning to end, with a large amount of heavy work. It was necessary to transport the heavy plant to the dam site at the earliest possible moment, and time was not available for the construction of such a line,

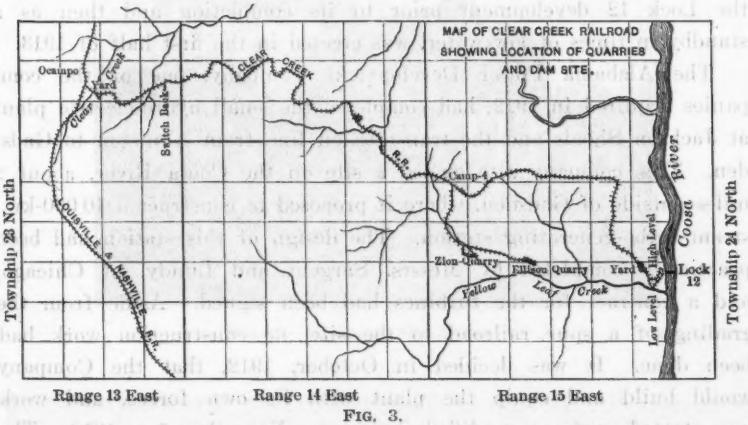


FIG. 3.

although the cost of the railroad as a unit would undoubtedly have been less. The construction and repair work on this railroad was started on August 12th, 1912, by the Company's forces, and on November 26th, the first car loads of the contractor's heavy plant were delivered at the dam site, at least 2 months earlier than would have been possible if an entirely new road had been built. Before this time, however, camps had been constructed at the dam, and preparations were being made for the installation of the plant. During the period when the plant was being set up, a coffer-dam, enclosing the area to be excavated for the power-house and a portion of the spillway, was constructed. This coffer-dam was unwatered for the first time on December 13th, 1912, and the excavation of the river bed was started.

Long delays occurred, due to heavy rains and floods, and it was not until April 12th that the first concrete was placed. In November, 1913, the power-house foundations having been prepared, the building of the superstructure and the installation of machinery were begun. All the concrete work of the power-house foundations and dam was completed on February 26th, 1914, 6 days before the limiting date set by the Government grant. The installation of the first unit was completed on March 22d, 1914. The completion of the installation of the other units followed within a short time thereafter. The first unit was tested out and dried and was put into commercial operation on April 12th, 1914, just 1 year after the first concrete was poured.

An auxiliary steam plant, to be used in building up a load for the Lock 12 development prior to its completion and then as a standby in times of low water, was erected in the first half of 1913.

The Alabama Power Development Company, one of the companies acquired in 1912, had completed the small hydro-electric plant at Jackson Shoals and the transmission line from Anniston to Gadsden. This company also owned a site on the Coosa River, about 2 miles outside of Gadsden, where it proposed to construct a 10 000-kw., steam turbo-generating station. The design of this station had been practically completed by Messrs. Sargent and Lundy, of Chicago, and a contract for the turbines had been signed. Aside from the grading of a spur railroad to the site, no construction work had been done. It was decided in October, 1912, that the Company would build and equip the plant with its own forces, and work was started under a modified design on November 1st, 1912. The plant was completed in June, 1913, but a delay in the delivery of the transformers prevented commercial delivery of power until July. Fig. 4 shows the exterior of the plant and Fig. 6 shows a section through the boiler and turbine rooms.

The construction of transmission lines and sub-stations was carried on simultaneously with the other work. The surveys for proposed transmission lines were started with three parties in July, 1912, the purchasing of right of way was begun in October, and actual construction was commenced in November. In March, 1914, the lines from Lock 12 to Birmingham (46.5 miles), from Lock 12 to Anniston, *via* Jackson Shoals (47.8 miles), from Sylacauga to Alexander City (25.1 miles), from Jackson Shoals to Leeds (26.5

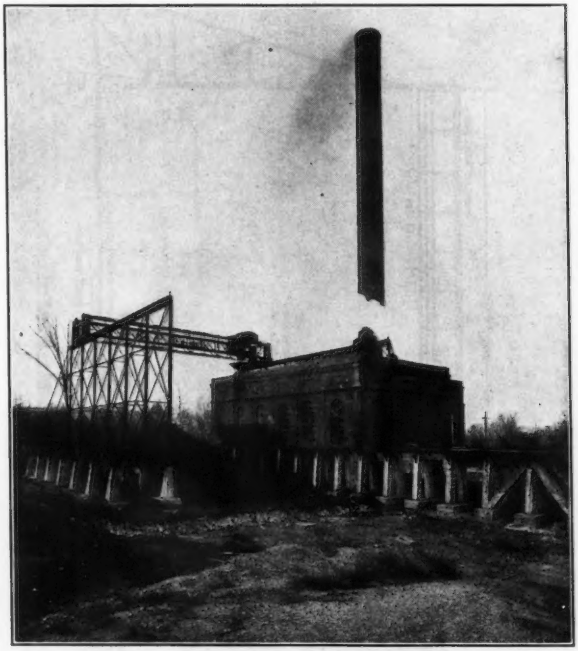


FIG. 4.—GADSDEN STEAM PLANT.

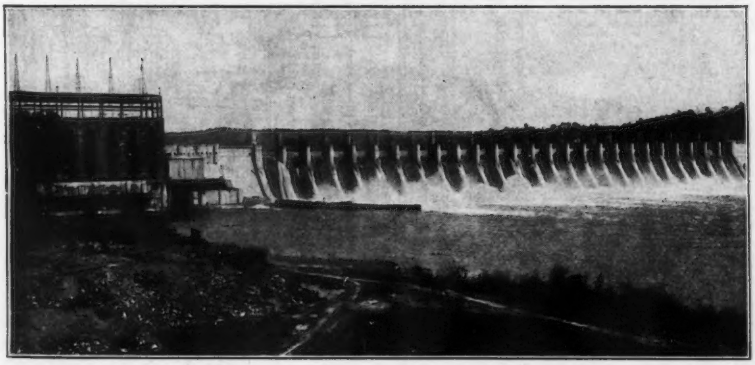
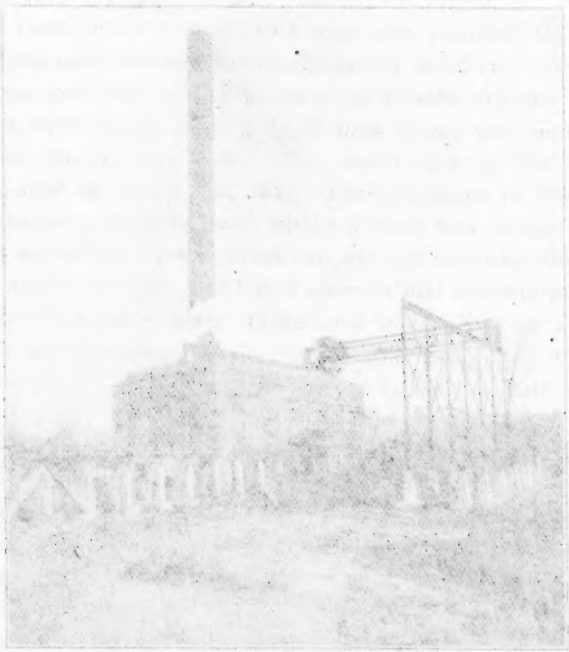
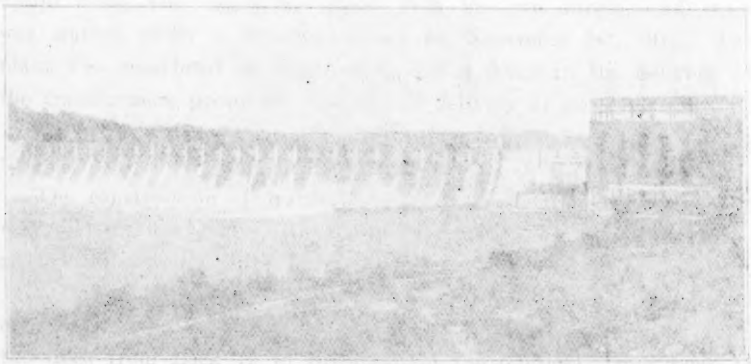


FIG. 5.—POWER-HOUSE UNDER CONSTRUCTION, BRICKWORK HAVING REACHED HIGH-TENSION FLOOR.





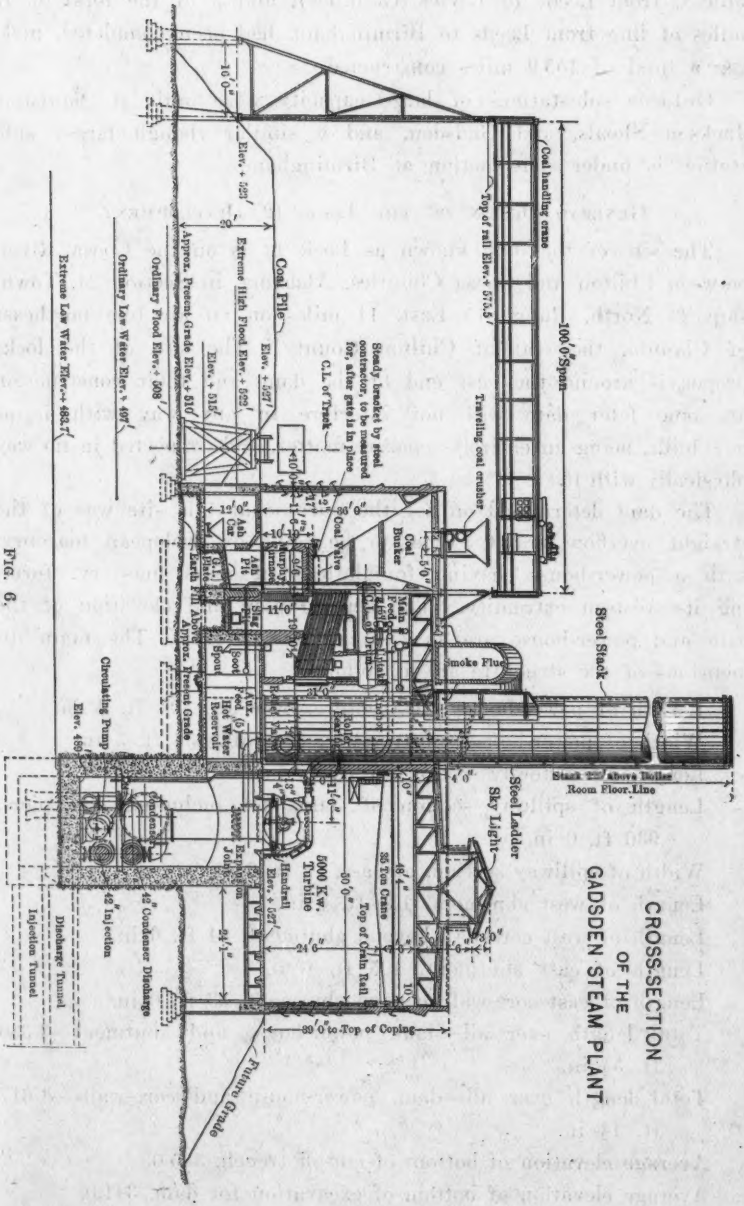


FIG. 6.—CROSS-SECTION OF THE GADSDEN STEAM PLANT

miles), from Leeds to Lovick (5 miles), and 5 of the total of 19 miles of line from Leeds to Birmingham, had been completed, making a total of 155.9 miles constructed.

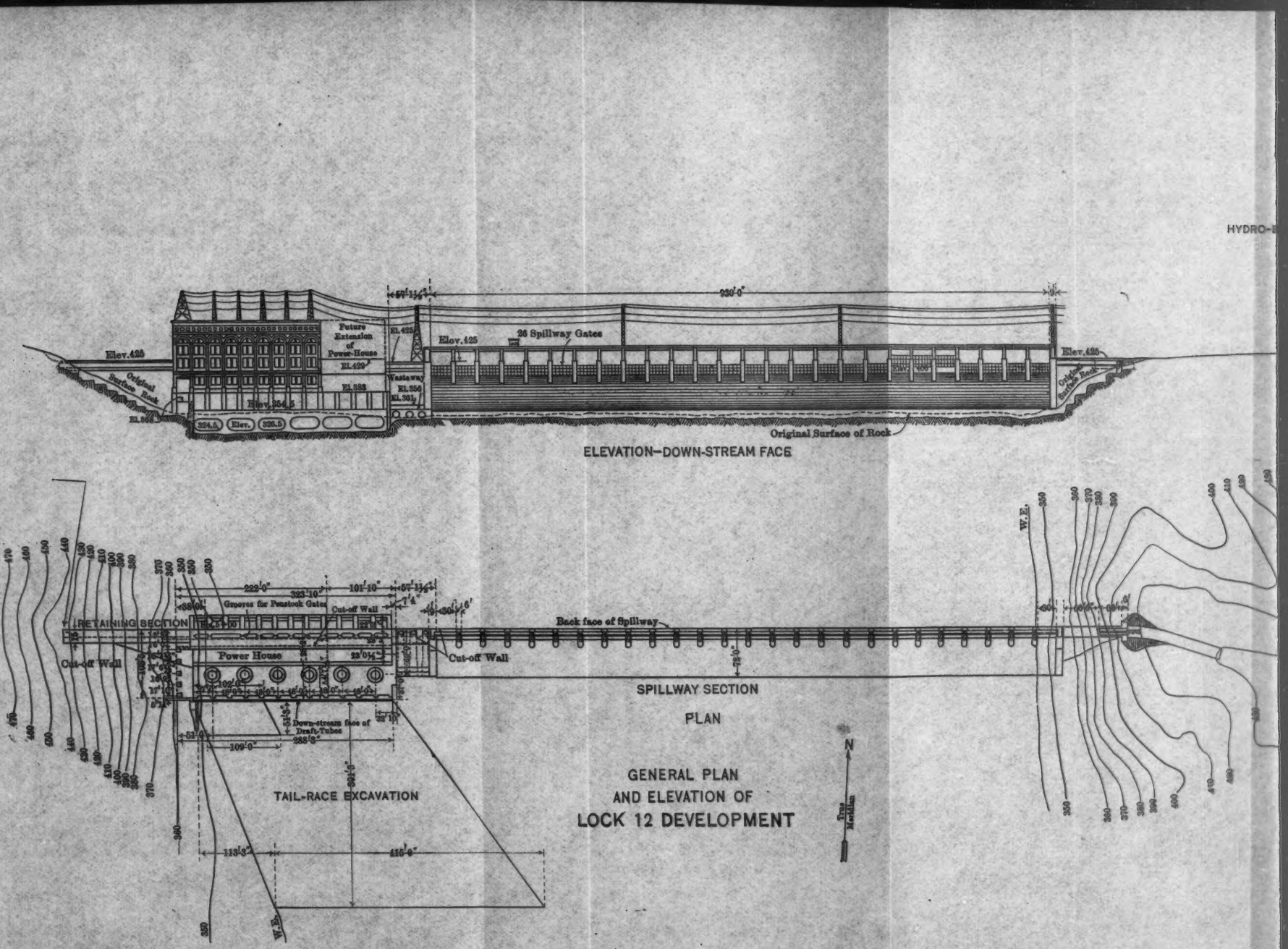
Outdoor sub-stations of large capacity were built at Anniston, Jackson Shoals, and Gadsden, and a similar though larger sub-station is under construction at Birmingham.

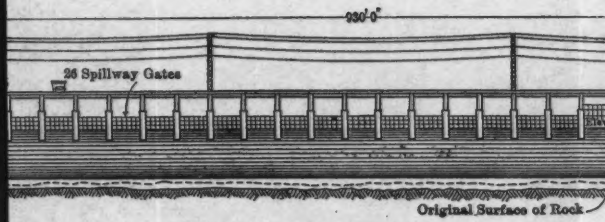
General Design of the Lock 12 Development.

The site of the dam known as Lock 12 is on the Coosa River, between Chilton and Coosa Counties, Alabama, in Section 24, Township 23 North, Range 15 East, 11 miles on an air line northeast of Clanton, the seat of Chilton County. The site of the locks proper is around the east end of the dam, and their construction at some later date will not interfere in any way with it as now built, being an entirely separate matter, and connected in no way physically with it.

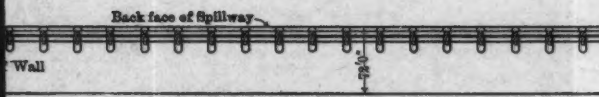
The dam determined on for the purpose at this site was of the straight overflow gravity type, to be built of cyclopean masonry, with a power-house, having foundations of mass masonry, forming its western extremity. The general plan and elevation of the dam and power-house are shown on Plate XXIII. The main dimensions of the structure are as follows:

Length of power-house substructure, over all, 303 ft. 4 in.
Width of power-house substructure, over all, 136 ft. 5 in.
Length of wasteway section, 57 ft. 1½ in.
Length of spillway section of dam, not including end piers, 930 ft. 0 in.
Width of spillway section, at base, 72 ft. 0 in.
Length of west abutment, 134 ft. 6 in.
Length of west core-wall beyond abutment, 34 ft. 6 in.
Length of east abutment, 105 ft. 6 in.
Length of east core-wall, beyond abutment, 53 ft. 0 in.
Total length over all—dam, power-house and abutment—1 530 ft. 5½ in.
Total length over all—dam, power-house and core-walls—1 617 ft. 1½ in.
Average elevation of bottom of cut-off trench, 335.0.
Average elevation of bottom of excavation for dam, 341.0.


Future Extension of Power-House
El. 429
Elev. 425
26 Spillway Gates
Elev. 425
Elev. 425
Original Surface Rock
Tailway
El. 356
El. 301
El. 383
Elev. 324.5
324.5
Elev.
326.5
Original Surface of Rock
ELEVATION—DOWN-STREAM FACE
RETAINING SECTION
Grooves for Penstock Gates
Cut-off Wall
Back face of Spillway
Cut-off Wall
Power House
Down-stream face of Draft-Tubes
SPILLWAY SECTION
PLAN
TAIL-RACE EXCAVATION
GENERAL PLAN
AND ELEVATION OF
LOCK 12 DEVELOPMENT
N
True Meridian
W.E.
W.E.



ELEVATION—DOWN-STREAM FACE

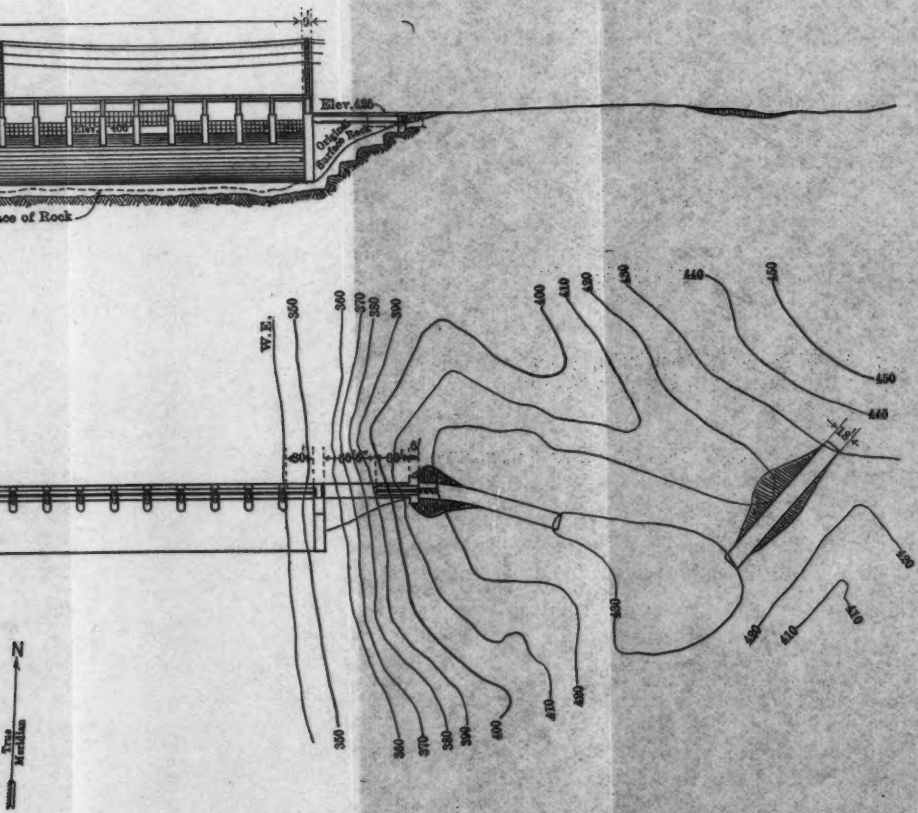


SPILLWAY SECTION

PLAN

GENERAL PLAN AND ELEVATION OF LOCK 12 DEVELOPMENT

PLATE XXIII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



Average elevation of bottom of river before excavation for dam, 343.0.

Elevation of crest of dam, 406.0.

Elevation of top of spillway piers, 444.0.

The spillway, forming practically all of the dam east of the power-house, has a net crest length of 780 ft., divided into 26 sections, each 30 ft. long, which are separated by piers 6 ft. long on the axis of the dam, the piers serving to carry flood-gates of the modified Stoney type.

At a point 11½ miles above Lock 12, the Louisville and Nashville Railroad crosses the Coosa on a through bridge of several trusses, and Peckerwood Creek, a tributary stream, immediately above this point, on a through plate-girder bridge. The elevation of the low steel on the truss bridge is 425.3 and of the plate girders 423.5. These elevations, allowing for the back-water effect which is exaggerated to some extent during floods by the Narrows (shown on Plate XXIV) about 5 miles below the dam, fix the normal elevation of the pond at 420, as established by the U. S. Army Engineers.

For the purpose of maintaining the normal pond level during floods, twenty-six flood-gates, each 14 ft. high, were designed to be placed on the crest of the spillway at Elevation 406. The abutment sections of the dam and the section between the power-house and the spillway in which are the wasteway culverts, are carried to Elevation 425, with parapet walls to Elevation 429.5.

The power-house is shown in cross-section by Fig. 7, on which it should be noted that the foundations are of mass concrete, the penstocks, scroll casings, and draft-tubes being moulded directly in the concrete with no lining of any kind.

One feature of this development, which is a decided improvement over most of those heretofore constructed, is that the units are entirely self-contained, that is, the turbine, generator, and exciter are self-supporting, and can be built complete, from the base ring of the turbine to the top of the direct-connected exciter, without requiring any lateral support from the concrete about the turbine casing during construction.

There are four units in the power-house, and provision in the foundations for two more. Each of the units has a capacity of

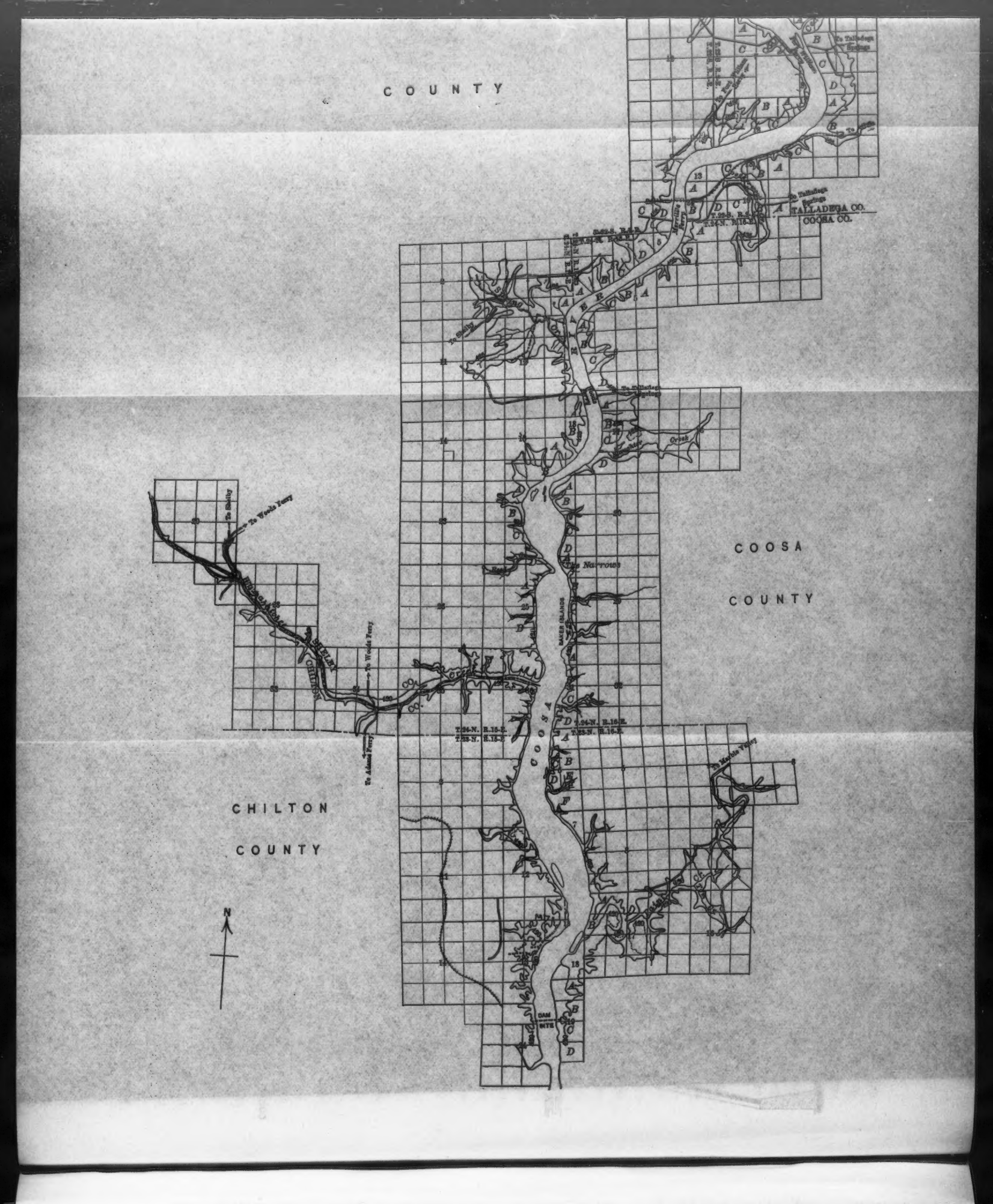
17 500 h.p. delivered to the shaft at a speed of 100 rev. per min., and under a head of 68 ft.

The adoption of this size of installation was dependent on many factors and a number of assumptions, a detailed statement of which is impossible here. It might be well, however, to note that the principal considerations involved were the ordinary working head, the reduction of head at times of flood, the nature and extent of the power market on which the load factor depends, the available steam auxiliary plants, and the relation of the plant at Lock 12 to the other proposed developments of the Company.

As noted previously, the year of lowest flow on record was 1904, when the discharge of the Coosa at Lock 12 dropped to 1 700 sec-ft. The storage of the reservoir, using a 10-ft. draw-down, increases the available draft to 2 300 sec-ft., which, with a 68-ft. head, will produce 14 200 h.p., of continuous, 24-hour power, having 80% efficiency. Combining this with the daily output of a steam plant with a capacity of 15 000 kw., there would be produced 614 000 kw-hr. per day, which, on a 50% load factor, is equal to 51 000 kw. maximum demand. A study of the horse-power percentage of time curve for Lock 12 shows that 70 000 h.p. on a 50% load factor, which is equal to 35 000 h.p., continuous, is available at Lock 12 for 80% of the time under a head of 68 ft. and 85% of the time under a head of 74 ft.

It should be noted that this is a pioneer development in Alabama, and that the character of the market is not as well defined as it is in most States where development has been going on for years. It was not known whether part of the load would be electro-chemical, or not, and it was necessary to assume the load factor on the best information obtainable from a hurried survey of the market.

It is estimated that the construction of the Etowah Reservoir at the head-waters of the Coosa, as previously mentioned, will increase the low-water flow of the river to 4 700 sec-ft. Assuming that Lock 12 would be operated ultimately in connection with this storage reservoir and the developments lower down the river, an ultimate installation of 105 000 h.p. is provided for in the power-house foundations.


COUNTY
TALLADEGA CO.
COOSA CO.
COOSA
COUNTY
CHILTON
COUNTY
COOSA
To Shelby
To Woods Ferry
To Adams Ferry
To Talladega Springs
To Marble Valley
BEELER
CREEK
BAKER ISLANDS
DAM SITE
T.24-N. R.15-E.
T.23-N. R.15-E.
T.24-N. R.16-E.
T.23-N. R.16-E.
N

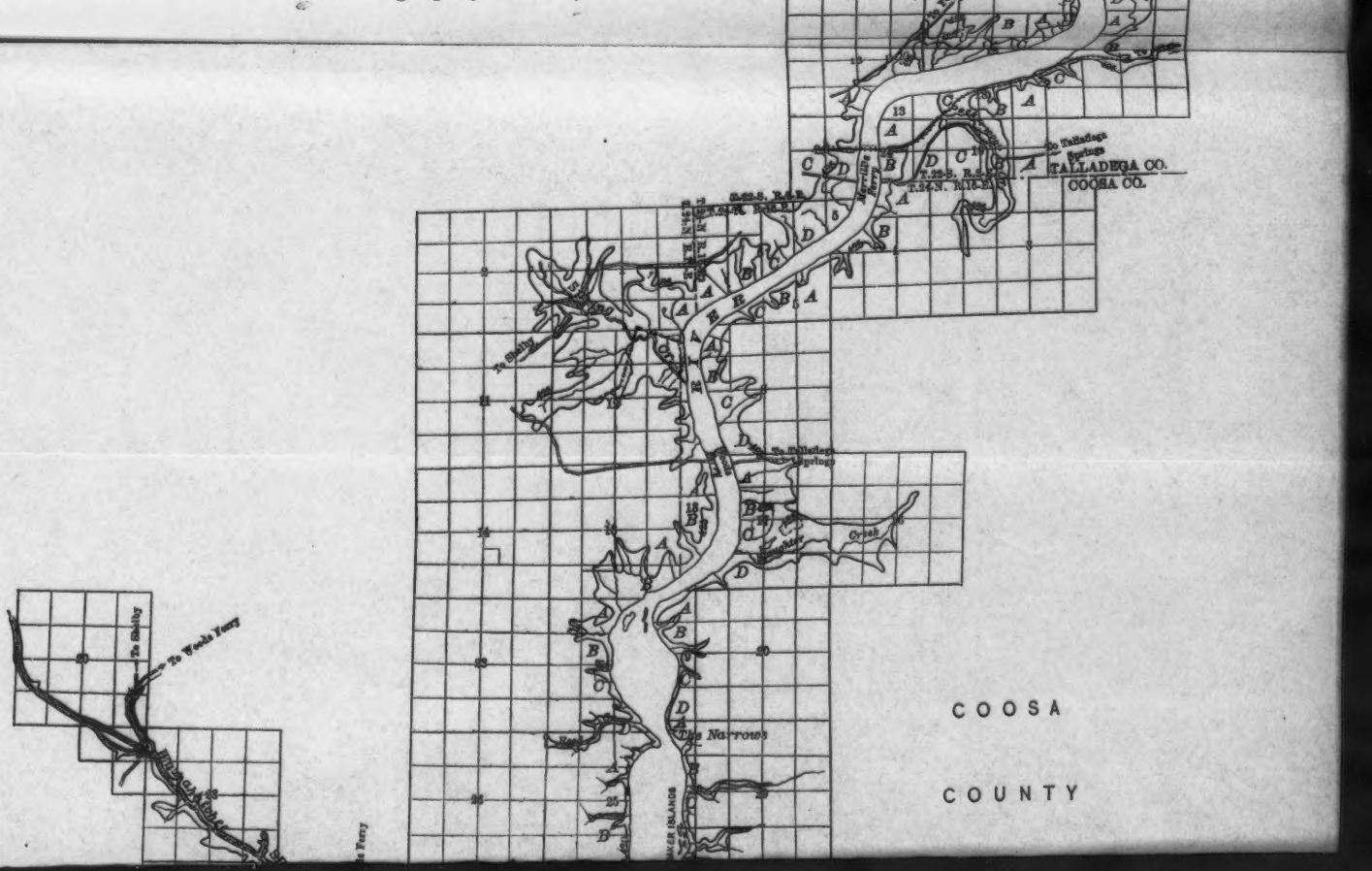
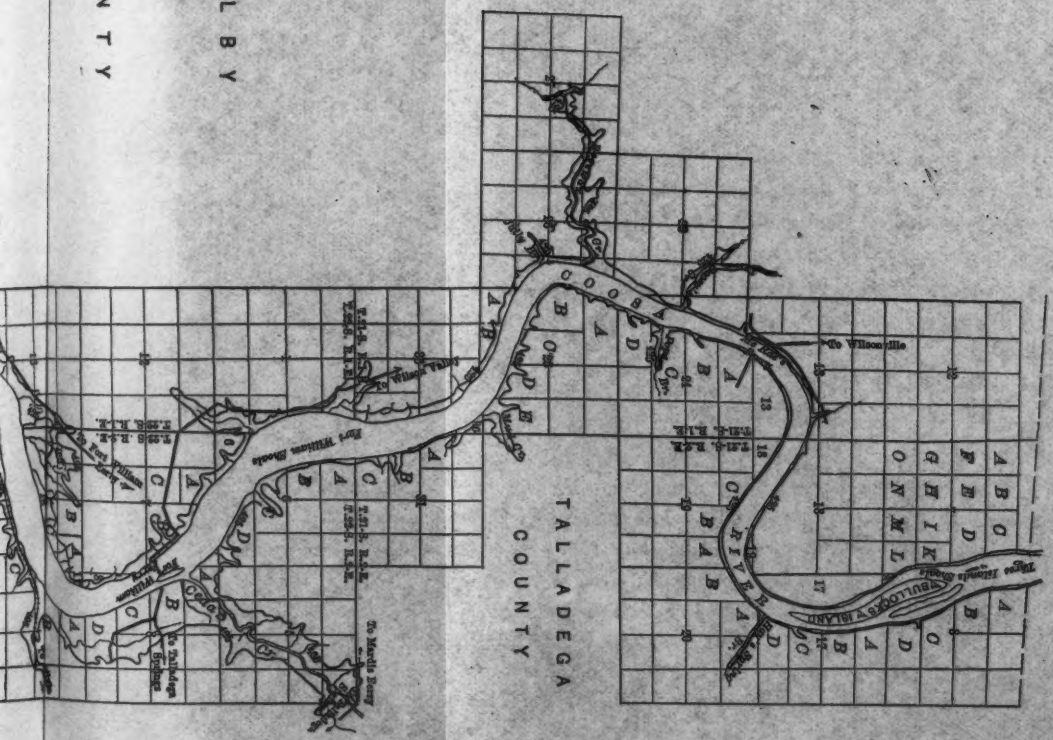

TALLADEGA CO.
COOSA CO.
To Talladega Springs
To Shilty
To Weeks Ferry
The Narrows
COOSA
COUNTY

PLATE XXIV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.

MAP OF LOCK 12 RESERVOIR



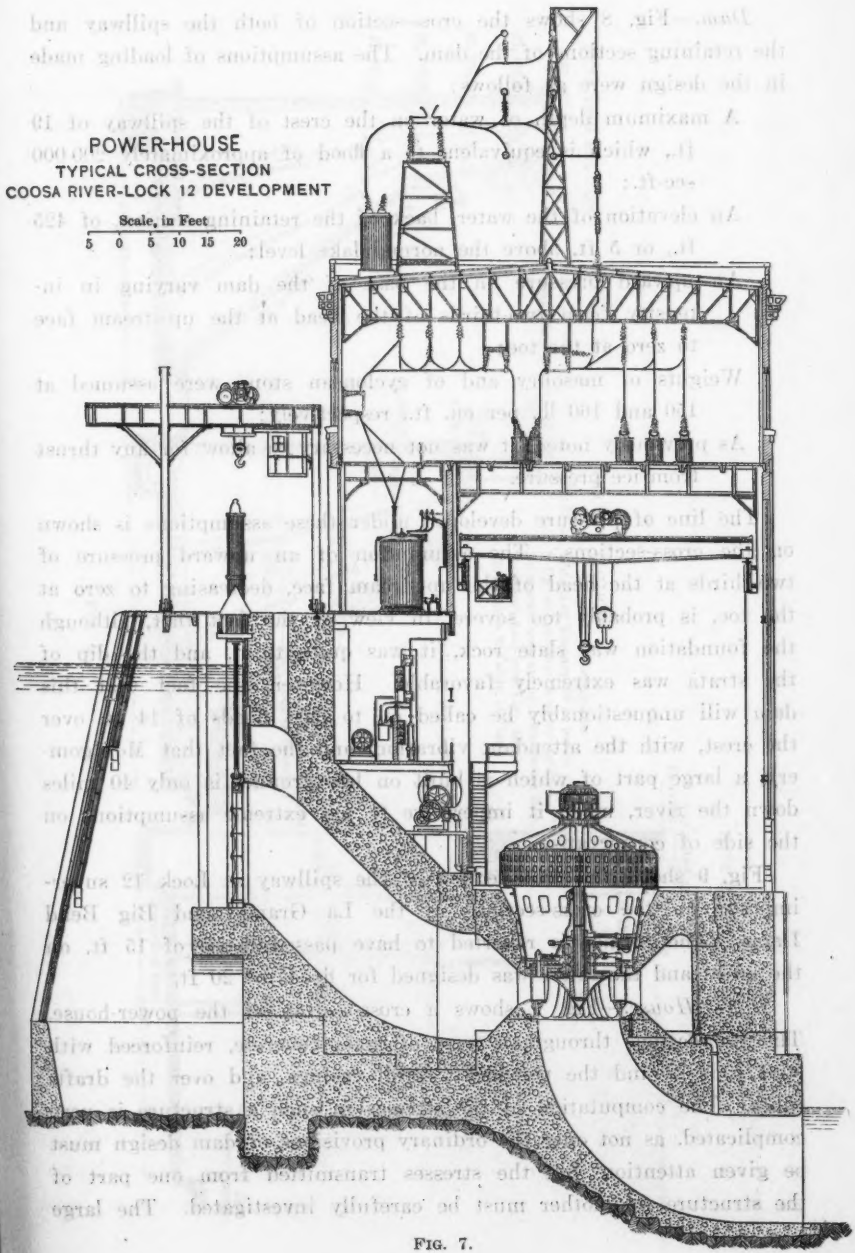

POWER-HOUSE
TYPICAL CROSS-SECTION
COOSA RIVER-LOCK 12 DEVELOPMENT
Scale, in Feet
5 0 5 10 15 20

FIG. 7.

Dam.—Fig. 8 shows the cross-section of both the spillway and the retaining sections of the dam. The assumptions of loading made in the design were as follows:

A maximum depth of water on the crest of the spillway of 19 ft., which is equivalent to a flood of approximately 200 000 sec-ft.;

An elevation of the water, back of the retaining section, of 425 ft., or 5 ft. above the normal lake level;

An upward pressure on the base of the dam varying in intensity from two-thirds of the head at the up-stream face to zero at the toe;

Weights of masonry and of cyclopean stone were assumed at 150 and 160 lb. per cu. ft., respectively;

As previously noted, it was not necessary to allow for any thrust from ice pressure.

The line of pressure developed under these assumptions is shown on the cross-sections. The assumption of an upward pressure of two-thirds at the head of the up-stream face, decreasing to zero at the toe, is probably too severe, in view of the fact that, although the foundation was slate rock, it was quite tight, and the dip of the strata was extremely favorable. However, the fact that this dam will unquestionably be called on to pass floods of 14 ft. over the crest, with the attendant vibration, and the fact that Montgomery, a large part of which is built on low ground, is only 40 miles down the river, made it imperative to use extreme assumptions on the side of conservatism.

Fig. 9 shows the cross-section of the spillway at Lock 12 superimposed on the cross-sections of the La Grange and Big Bend Dams. The former is reported to have passed floods of 15 ft. on the crest, and the latter was designed for floods of 20 ft.

Power-House.—Fig. 7 shows a cross-section of the power-house. The foundations throughout were of mass concrete, reinforced with steel rods around the penstocks, scroll casings, and over the draft-tubes. The computation of the stresses in such a structure is very complicated, as not only the ordinary provisions of dam design must be given attention, but the stresses transmitted from one part of the structure to another must be carefully investigated. The large

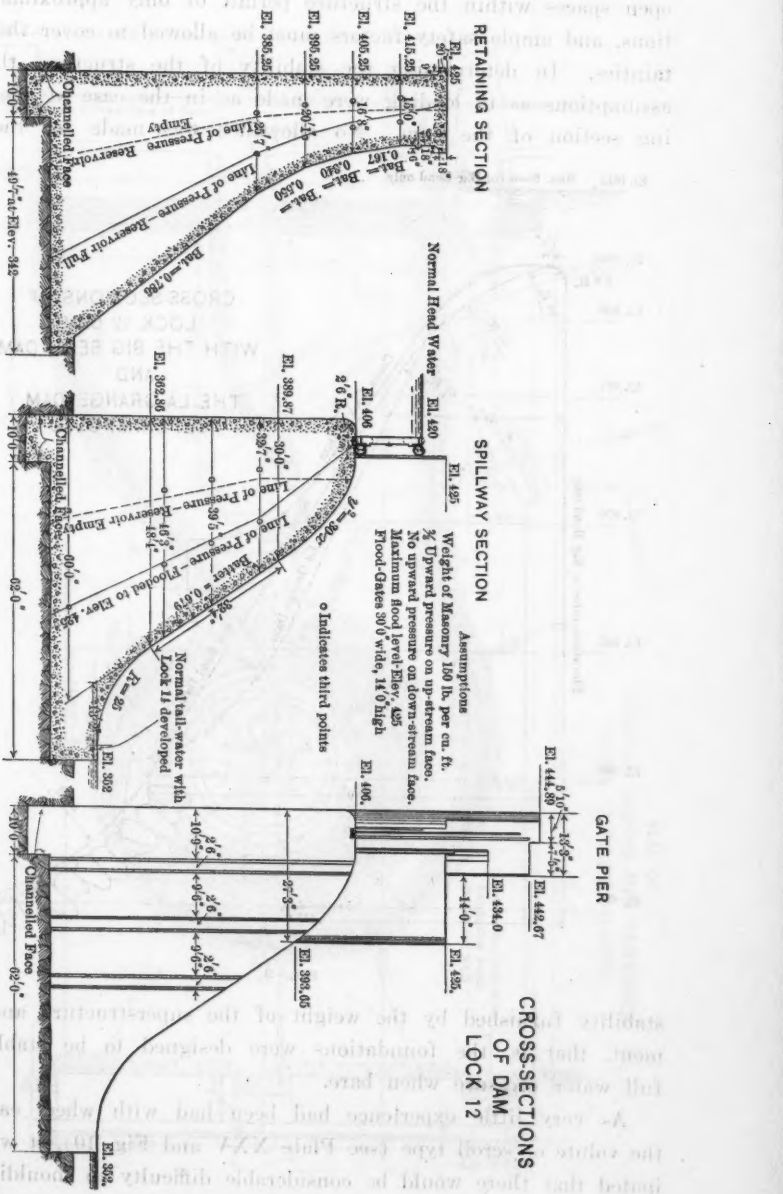

RETAINING SECTION
SPILLWAY SECTION
GATE PIER
CROSS-SECTIONS OF DAM LOCK 12
Normal Head Water El. 430
Assumptions
Weight of Masonry 150 lb. per cu. ft.
No upward pressure on up-stream face.
Maximum flood level-Elev. 425
Flood-Gates 30'0" wide, 14'0 high
o Indicates third points
Normal tail-water with Lock 11 developed
Channelled Face
Empty
Line of Pressure – Reservoir Full
Line of Pressure – Reservoir Empty
Line of Pressure – Flooded to Elev. 425
Batter = 0.67½
Bat.=0.786
Bat.=0.167
Bat.=0.340
Bat.=0.450
El. 425
El. 415.25
El. 405.25
El. 395.25
El. 390.25
El. 389.87
El. 382.86
El. 406
El. 402
El. 444.80
El. 442.67
El. 434.0
El. 425
El. 393.05
10'-0" at Elev. 342
62'-0"
27'-9"
30'-0"
14'-9"
2'-6" R.
R = 25'-0"
R = 30'-0"

Fig. 8.

open spaces within the structure permit of only approximate solutions, and ample safety factors must be allowed to cover the uncertainties. In determining the stability of the structure, the same assumptions as to loading were made as in the case of the retaining section of the dam. No allowance was made for the added

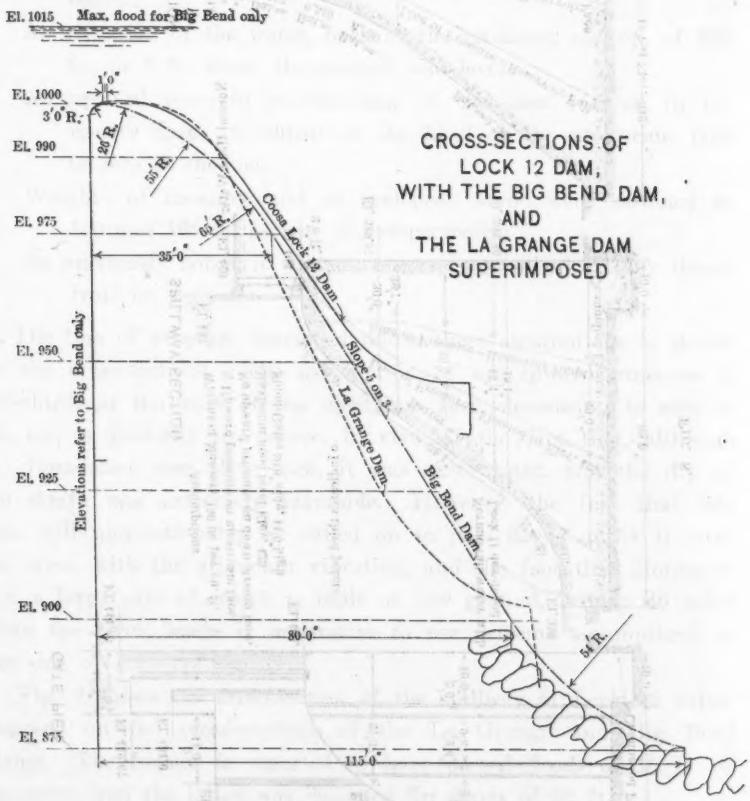
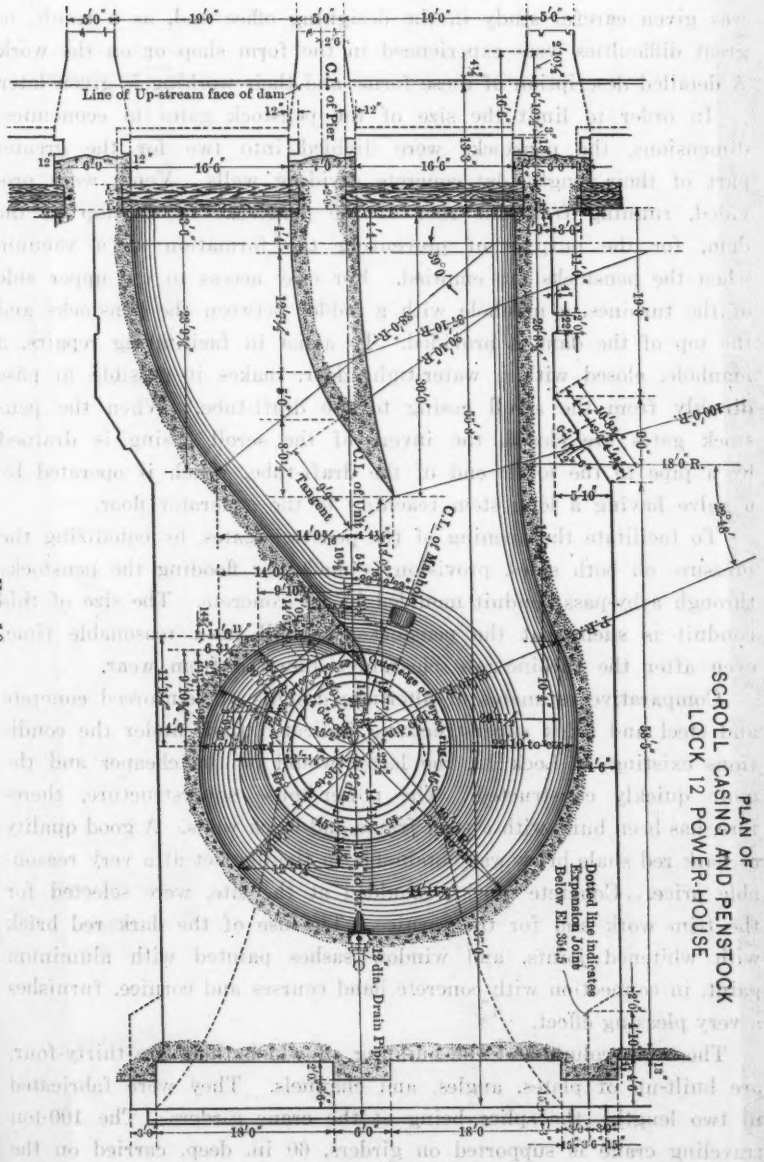


FIG. 9.

stability furnished by the weight of the superstructure and equipment, that is, the foundations were designed to be stable under full water pressure when bare.

As very little experience had been had with wheel casings of the volute or scroll type (see Plate XXV and Fig. 10), it was anticipated that there would be considerable difficulty in moulding them



PLAN OF SCROLL CASING AND PENSTOCK LOCK 12 POWER-HOUSE

FIG. 10.

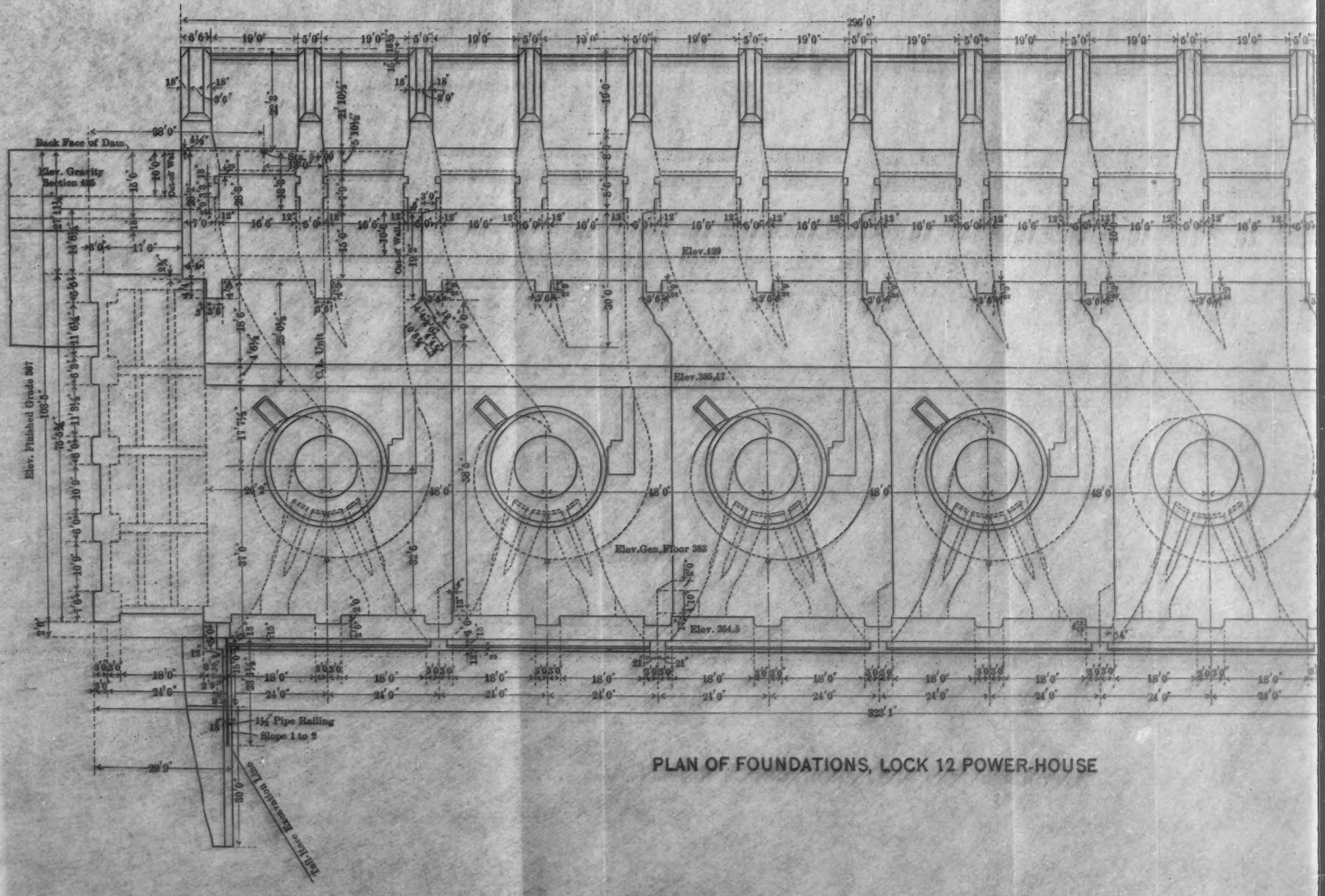
directly in the concrete. For this reason, the structure of the forms was given careful study in the designing office and, as a result, no great difficulties were experienced in the form shop or on the work. A detailed description of these forms and their working is given later.

In order to limit the size of the penstock gates to economical dimensions, the penstocks were divided into two for the greater part of their lengths by concrete dividing walls. Vents were provided, running from the roof of the penstocks to the top of the dam, for the purpose of preventing the formation of a vacuum when the penstocks are emptied. For easy access to the upper side of the turbines, a manhole with a ladder between the penstocks and the top of the dam is provided. To assist in facilitating repairs, a manhole, closed with a water-tight door, makes it possible to pass directly from the scroll casing to the draft-tube. When the penstock gates are closed, the invert of the scroll casing is drained by a pipe to the lower end of the draft-tube, which is operated by a valve having a long stem reaching to the generator floor.

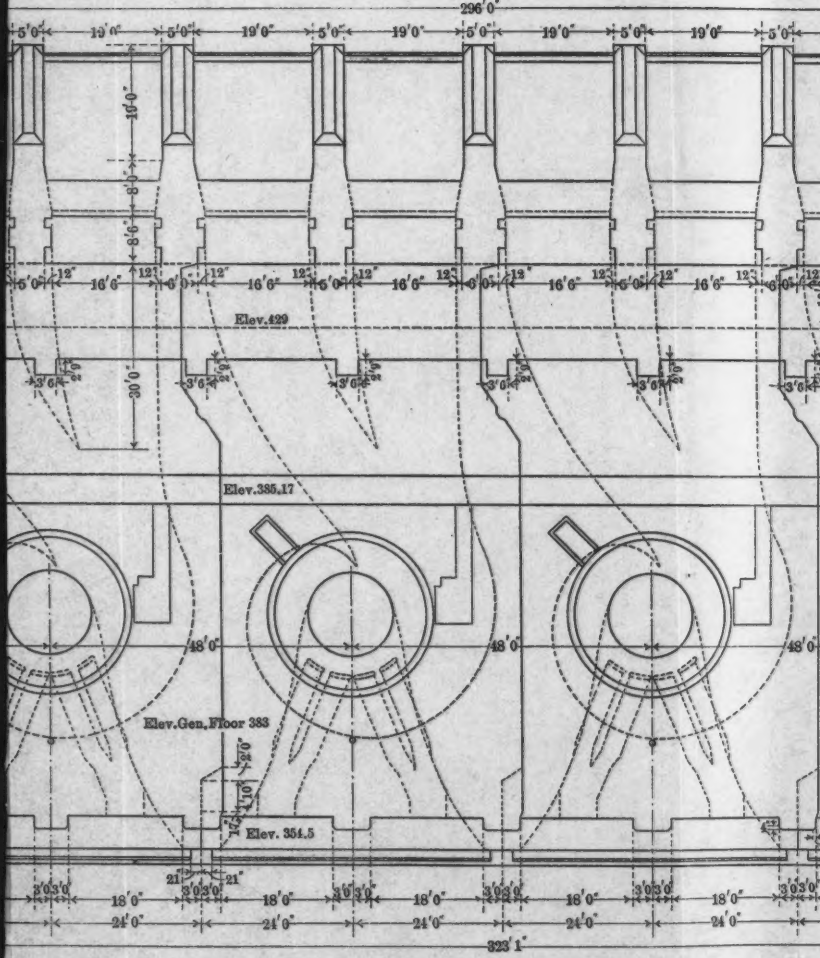
To facilitate the opening of the penstock gates, by equalizing the pressure on both sides, provision is made for flooding the penstocks through a by-pass conduit moulded in the concrete. The size of this conduit is such that the penstocks will fill in a reasonable time, even after the turbine-gate clearances increase from wear.

Comparative estimates of alternate designs of reinforced concrete and steel and brick superstructures indicated that, under the conditions existing at Lock 12, the latter would be the cheaper and the more quickly constructed. The power-house superstructure, therefore, has been built with a steel frame and brick walls. A good quality of dark red shale brick was obtainable in the district at a very reasonable price. Concrete blocks, moulded at the site, were selected for the trim work and for the cornice. The use of the dark red brick with whitened joints, and window sashes painted with aluminum paint, in connection with concrete band courses and cornice, furnishes a very pleasing effect.

The main columns of the building, of which there are thirty-four, are built-up of plates, angles, and channels. They were fabricated in two lengths, the splice being at the crane girders. The 100-ton traveling crane is supported on girders, 60 in. deep, carried on the main columns of the building. The roof trusses have a clear span

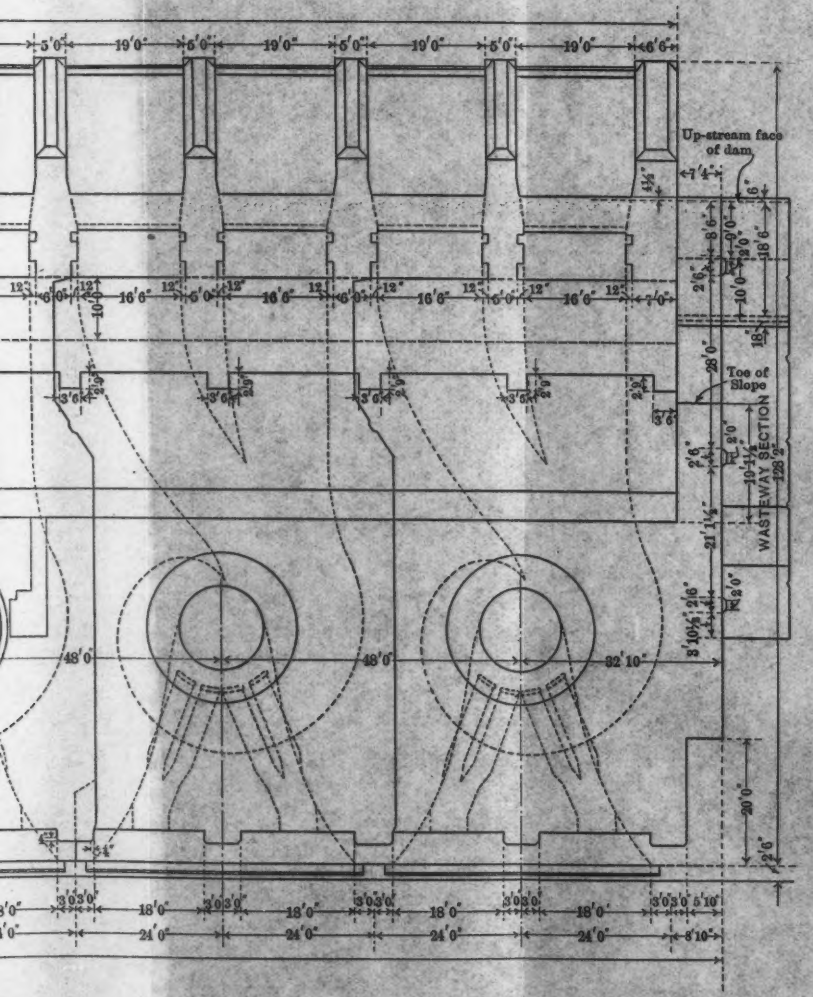


PLAN OF FOUNDATIONS, LOCK 12 POWER-HOUSE



PLAN OF FOUNDATIONS, LOCK 12 POWER-HOUSE

PLATE XXV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



HOUSE

of 69 ft., are 9 ft. deep at the mid-point, and, in addition to supporting themselves and the dead load of the roof, are designed to carry, attached to their lower chords, the high-tension bus structure of the power-house, and, on top of the roof, the electrolytic lightning arresters and several outgoing line towers.

The floors of the power-house superstructure are of reinforced concrete slabs supported on steel beams.

Turbines.—The turbines selected for this installation are the largest capacity, vertical, single-runner turbines ever installed in the United States, to operate under a head of approximately 70 ft.

Fig. 11 is a cross-section through a complete unit, and Figs. 12 and 13 show the runner and the pit liner, respectively. The weight of a runner and shaft complete is 103 000 lb., and the weight of a complete turbine is 520 000 lb.

As previously mentioned, one feature in which these turbines differ from most of those heretofore installed is that they are self-contained. The bottom of the pit liner rests directly on the concrete foundation at the scroll-casing level, where anchorages make it possible to adjust the level of any point on it. This arrangement makes it possible to set up the turbine complete in the shop and do all the necessary machine work there, eliminating the very expensive machine work commonly done in the field. The installation of the turbine on its permanent foundation then becomes a comparatively simple matter, involving only the accurate setting of the base ring on the anchorages and the bolting up of the other sections of the pit liner to this ring. If the machine work in the shop has been accurate, the assembly in the field must be simple. The runner of these units is of cast iron and of the Francis inflow type, cast in one piece. The diameter of the runner, over all, is 13 ft. 3 in., and it is 7 ft. 2 in. high.

The casing or pit liner is made up of three rings; a heavy cast-iron ring, in two sections, forms a foundation for the rings above and transmits the weight of the generator and moving parts from them to the concrete foundation; it also serves the purpose of guiding the water from the scroll casing to the movable guide-vanes. This ring of the pit liner is commonly called the speed ring.

Above the speed ring is the intermediate ring. This serves to transmit the load to the speed ring, to form a water-proof lining for the operating chamber over the runner, and to carry the two gate-

operating engines of the turbine. Above this is the third section of the pit liner, the foundation ring, which rises to the elevation of the main floor of the power-house. On this ring is set the generator armature or stator, and the weight of the latter and of all moving parts is transmitted through it. Bolted to this ring, and forming part of it, is

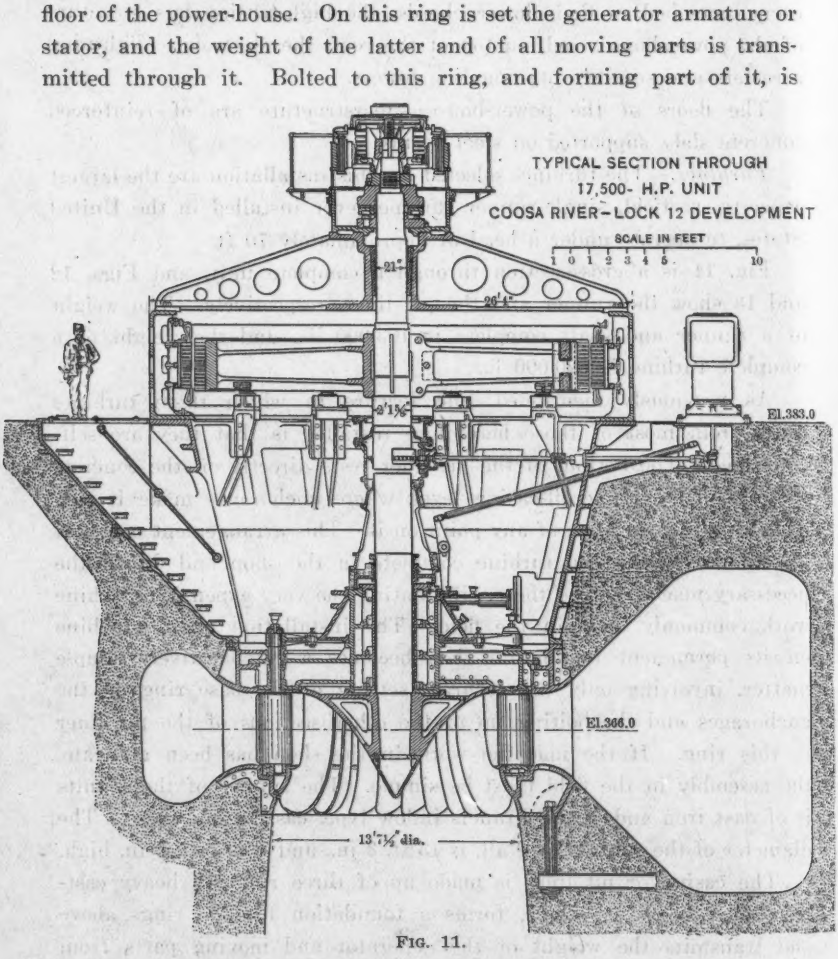


FIG. 11.

the cast-iron stairway giving entry from the generator floor to the operating chamber. The floor of the operating chamber is the cover-head, which is between the speed ring and the intermediate ring of the pit liner.

The shaft, 24 in. in diameter, is one solid piece, passing throughout the turbine, generator, and exciter. The weight of the shaft

FIG. 12.—RUNNER OF LOCK NO. 12 TURBINES IN SHOP.

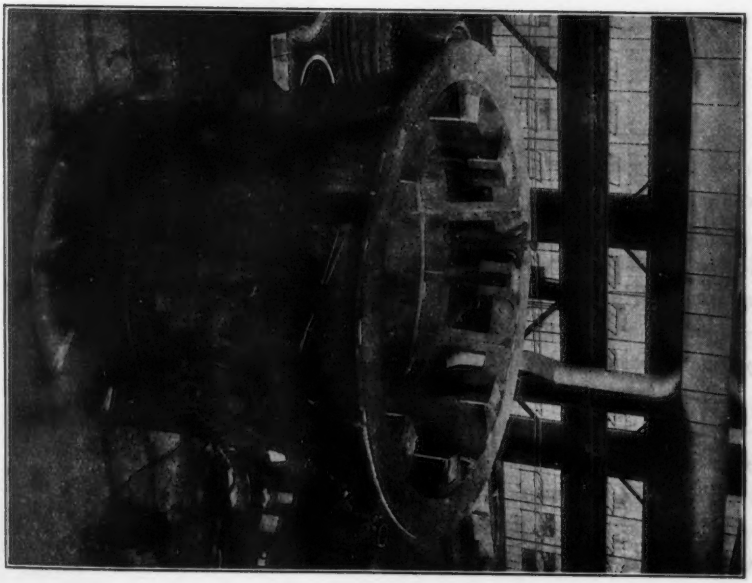
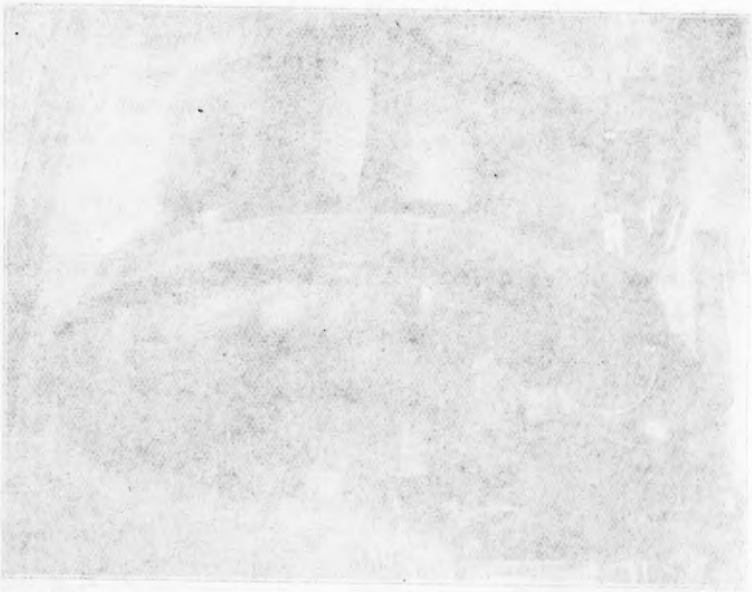
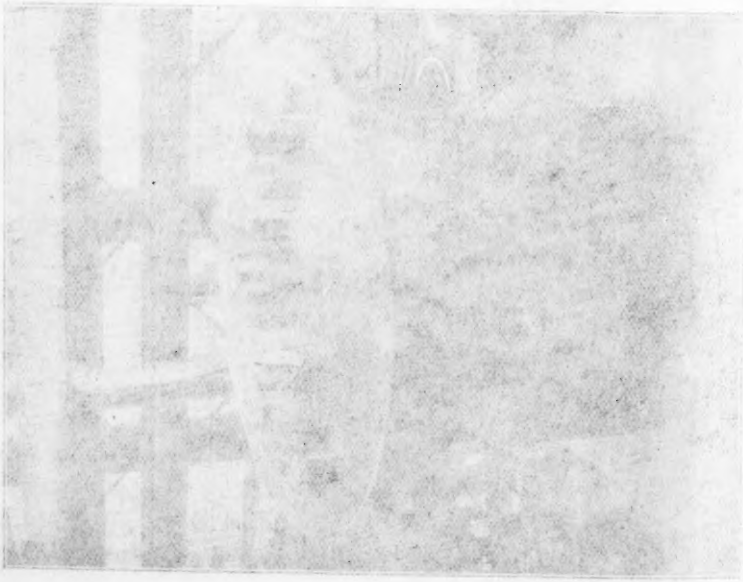


FIG. 13.—COMPLETE PIT LINER OF TURBINE ASSEMBLED IN SHOP.





and the running parts attached to it is transmitted through a thrust-bearing, immediately above the generator and below the exciter, to the bridge on the generator. The runner is made a taper fit on the shaft, the torsion being taken on a long vertical key. A circular key and a cast-iron cap hold the runner in place. The shaft is held in position above the runner by lignum-vitæ guide-bearings in the cover-head of the turbine and by a Babbitted steady bearing under the bridge of the generator. The former is lubricated with water, and the latter with oil. The length of contact of the lignum-vitæ guide-bearings is 70¼ in.

The turbines in the plant are guaranteed by the manufacturers to deliver at the shaft 17 500 h.p., with a head of 68 ft. and a speed of 100 rev. per min.

The runner selected was stepped up from a model which, on a Holyoke test, developed a maximum efficiency of 89.68%, and the manufacturers have guaranteed that the units as installed will have a maximum efficiency of 87% when delivering to the shaft from 15 000 to 17 000 h.p. The efficiencies guaranteed at full and part gate are as follows:

Power.	Efficiency.
17 500 h.p.....	83 per cent.
15 300 "	85½ "
13 125 "	83 "
8 750 "	74 "

The efficiencies obtained in the Holyoke tests are higher than have ever been attained on any turbine heretofore constructed, regardless of speed, capacity, or head.

No opportunity to check the actual efficiencies of the installed units will present itself until the next low-water season, in the summer or fall of 1914, and it is to be hoped that those in charge of the operation of the plant at that time will present a full account of the tests, as they should prove of great interest to the Profession.

These questions received much study, in connection with this development, and it was decided to use vertical units principally because it is possible to obtain higher efficiencies with large capacity vertical than with horizontal units. The reason for this is that in the draft-chests of large horizontal units there are of necessity sharp bends

causing losses of head, and there are other losses from the confluence of streams in the draft-tubes of multi-runner horizontal units. The ideal draft-tube is obtainable with the single-runner, vertical turbine, in which the water is received from the runner at high velocity and conducted away with gradually decreasing velocity to the tail-race, with small loss of effective head from friction or other causes. Volute or scroll casings which, in the last few years, experience has shown to return the highest efficiencies, are impossible to obtain economically in large capacity horizontal units. Besides these considerations, there are other objections to horizontal units in large capacity low-head installations, such as the facts that the unit would be below the level of tail-water, under flood conditions the power-house required would have to be larger, and the structural features of the horizontal shafts for these large wheels would not be satisfactory.

With regard to single *versus* multi-runner vertical units, the following points deserve mention: The ideal scroll casing, by which the inflowing water is brought to the runner at a uniform velocity at all points, cannot be obtained with multi-runner turbines. The effect of multi-runner turbines on draft-tubes is to cause either sharp bends, thereby decreasing the efficiency, or, in an effort to avoid this, require the lengthening of the unit to uneconomical proportions.

The single-runner unit makes possible an ideal arrangement of the operating mechanism, by which it is at all times open to inspection in the operating chamber above the runner. Other advantages of this type are that it is possible to lubricate the gate stems with grease cups, and that there are fewer parts to get out of order.

The selection of speed involves a close study of the inter-relations of power, efficiency, and cost per horse-power developed. In general, other things being equal, the cost of turbines per horse-power developed decreases as the speed increases, the horse-power increases as the speed increases, and the efficiencies at part gate fall off greatly as the speed increases.

An illustration of this last consideration is shown by Table 5, which is a comparison of the efficiencies of these runners at different speeds, under a head of 68 ft. for this installation.

For this development the runner developing 18 500 h.p. at a speed of 100 rev. per min., was selected because a great increase in power was obtainable over the lower-speed runner, with only a small sac-

rifice of efficiency under full load, and the half-load efficiency held up well above that of the higher-speed runner. The consideration that the head would decrease in time of flood to 56 ft. argued against the adoption of a runner with a half-load efficiency as small as 73 per cent. It should be noted that these efficiencies and capacities are not the same as those guaranteed for the full-sized runner, but are those developed from Holyoke tests on model runners.

TABLE 5.

	Efficiency at:		
	94.8 rev. per min.	100 rev. per min.	120 rev. per min.
Developing full power	90.6%	89.5%	90.1%
Developing half power	80.0%	77.0%	73.0%
Full power	16 850 h.p.	18 500 h.p.	17 500 h.p.*

* This runner was smaller than either of the other two, which accounts for the lower power than at 100 rev. per min.

Thrust-Bearings.—The function of the thrust-bearing, as previously noted, is to transmit to the bridge of the generator frame the weight of all the rotating parts and, in addition, the water thrust. Prior to 1912, roller- and step-bearings were the types in general use in hydraulic turbines. As its name signifies, the roller-bearing depended on a nest of rollers to reduce the friction while transmitting heavy loads in motion. In some forms of step-bearings, the load is carried by hydraulic or oil pressure at the foot of the shaft. A combination of these two types is in use in some plants. In 1912 a bearing of an entirely different design, which previously had been introduced into steam turbine practice, was applied to the hydraulic turbine. In October, 1912—when the selection of thrust-bearings was taken under consideration—there were only thirty of these Kingsbury bearings in use, and they were mostly in steam turbines under unit pressures of from 300 to 500 lb. per sq. in. of bearing surface. One was in use at the McCall's Ferry Plant and one additional had been ordered. Fig. 14 shows this bearing.

The lower casting with the pockets is stationary, and is bolted fast to the bridge of the generator frame. The segmental castings, with the flat Babbitted surface and the circular pocket on the underside, are the bearing blocks or shoes. These bearing blocks are sup-

ported on disks, the upper surface of which is a sphere of large radius, the disks fitting into the pockets under the bearing blocks. A collar, securely fastened to the shaft and accurately machined on the lower surface, rests on the bearing blocks when the unit is completely assembled. As the bearing blocks rest on a spherical surface, they are free to turn slightly as if on a pivot. When ready for operation, the interior parts of the bearing are completely immersed in oil. The fundamental principle of operation of the bearing, as expressed clearly by the inventor, is "perfect automatic lubrication effected by the pivotal support of the stationary bearing blocks or 'shoes,' which permits the formation of an oil film, completely separating the sliding surfaces."

This bearing seemed to do away with the objectionable features of the roller-bearing, namely, large friction losses, high initial cost, inaccessibility, and wear of the moving parts.

The bearing placed in June, 1912, in a 10 000-kw. unit at McCall's Ferry, was under a load of 410 000 lb. at 94 rev. per min. (equivalent to a unit pressure of 350 lb. per sq. in. of bearing surface) and was inspected in October, 1912, after 3½ months of steady service. The Babbitted faces of the shoes had been scraped before being placed, and they still showed distinctly the marks of the scraper. The area of contact, as shown by the bright area of the Babbitt, had extended but slightly, showing that the wear had been negligible. A computation made from the rise of temperature of the oil supplied to the bearing indicated a coefficient of friction of 0.0008. The bearing is made accessible for inspection of the Babbitted surfaces by wedges under the shoes, several of which may be removed at once, without lifting the shaft. The bearings, as made for the McCall's Ferry Plant, had an outside diameter of 48 in. The shaft diameter is 21 in.

After a careful consideration of the claims for the Kingsbury bearing and an inspection of that in service at McCall's Ferry, it was decided to adopt it. As constructed, it is 42 in. in outside diameter and supports a load of 330 000 lb.

A duplicate oiling system has been provided for the lubrication of these bearings. There are two tanks, each with a capacity of 838 gal., on the high-tension floor, from which the oil flows by gravity to the casing of the bearing. Provision has been made for the supply of 15 gal. per min. to the bearings, but this quantity will be regulated as

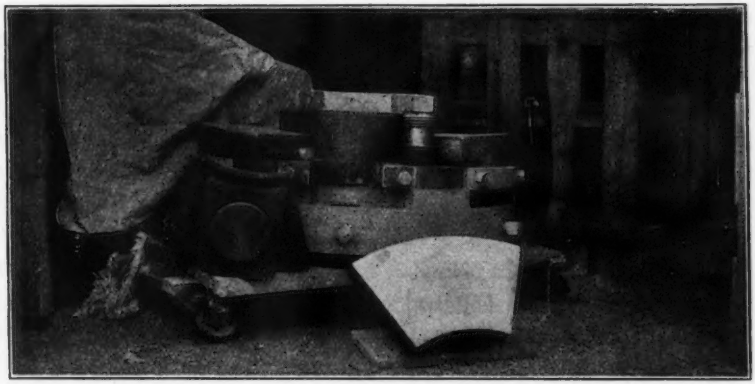
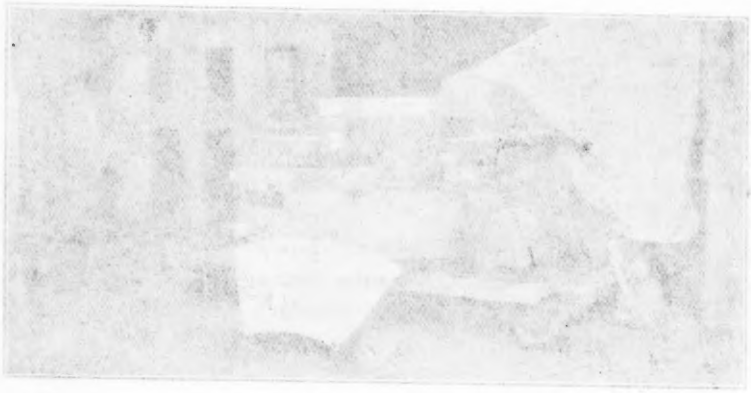
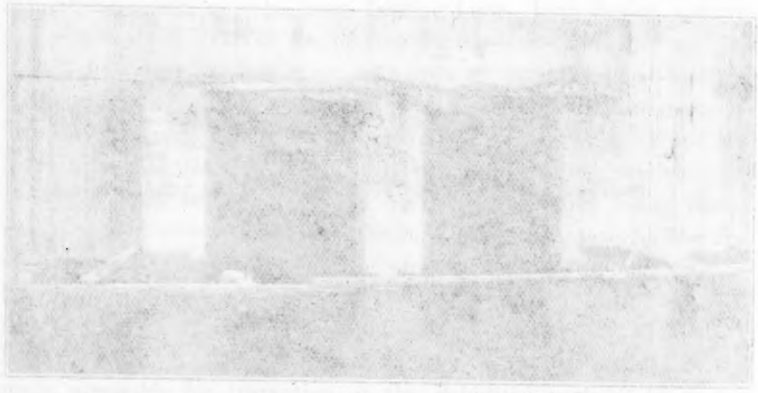


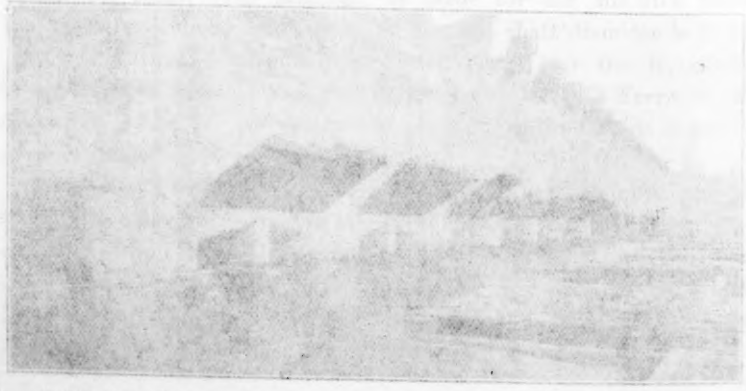
FIG. 14.—KINGSBURY BEARING.

FIG. 15.—TYPE OF TIMBER SLIDE-GATES USED TO CLOSE SEVERAL OF THE STREAM-CONTROL CULVERTS.

FIG. 16.—STREAM-CONTROL CONDUITS THROUGH DAM, WITH FLAP-GATES IN POSITION FOR CLOSING.







experience develops the best method of working the system. The temperature of the oil at the entrance to and exit from the bearing may be noted with the thermometers provided, so that the supply may be regulated. On leaving the bearing, the oil is pumped back through tanks, in which there are cooling coils, to the tanks on the high-tension floor. These coils are 2 ft. in diameter, are made up of 32 turns of 1½-in. pipe, and have a cooling surface of 105 sq. ft. No experience was available relative to the desirable capacity of these coils; their design is based on judgment alone.

Generators.—The four generators installed are 13 500 kv-a., 6 600-volt, 3-phase, 60-cycle generators, to operate at 100 rev. per min. The generator stator is mounted directly on the foundation ring of the turbine, to which it transmits the load received from the thrust-bearing.

The ventilation of the machine is effected by taking, through ducts in the concrete foundation, fresh air from outside the power-house and circulating it by the fan action of the rotor through the armature and out through holes left in the stator frame into the body of the power-house. The ducts provided for the purpose have a capacity of 60 000 cu. ft. of air per minute at a velocity of 2 000 ft. per min. The efficiencies guaranteed by the manufacturers are as follows:

Full load	95.7 per cent.
Three-quarter load	95.0 " "
One-half "	93.5 " "

Exciters.—Mounted above the thrust-bearing, and securely bolted to the main shaft, is the exciter. These exciters are of the shunt-wound type, with interpoles. The field frame rests on the generator bridge. They are rated at 150 kw., at 100 rev. per min., and 250 volts. An auxiliary motor generator exciter of 150 kw., 250 volts, is provided.

Governors.—The governors are four, modified Keokuk type, actuators, with independent motor-driven pumps, and tanks for the control of the double servo-motors furnished with the turbines. This apparatus is guaranteed, at 200 lb. per sq. in. pressure, to open or close the turbine gates completely in 2 sec., if not called on to develop a greater energy than 250 000 ft-lb. The governors are also

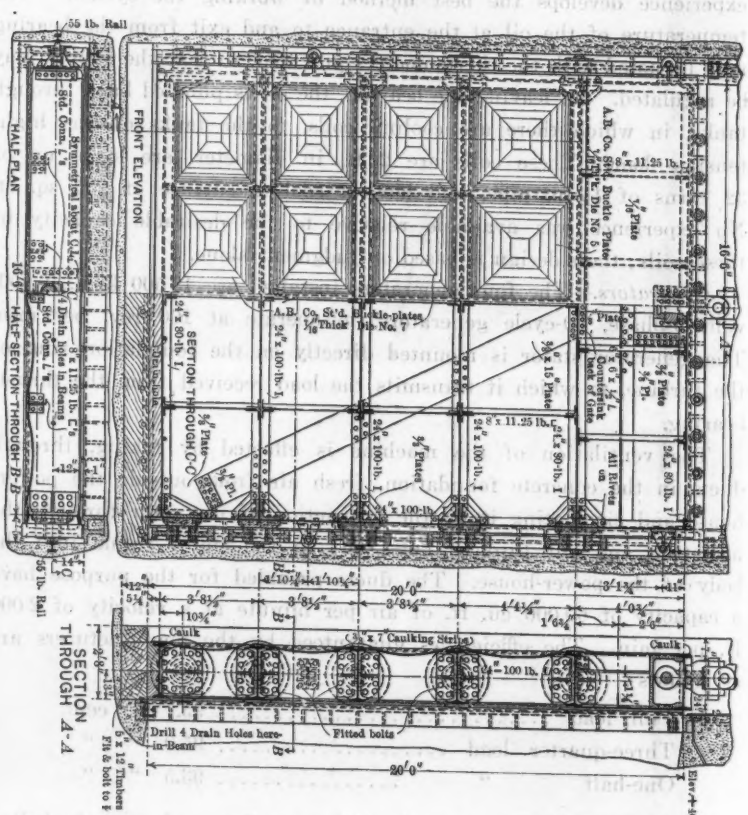


FIG. 17.

PENSTOCK GATES WITH SPRING SEALING DEVICE LOCK 12 DEVELOPMENT

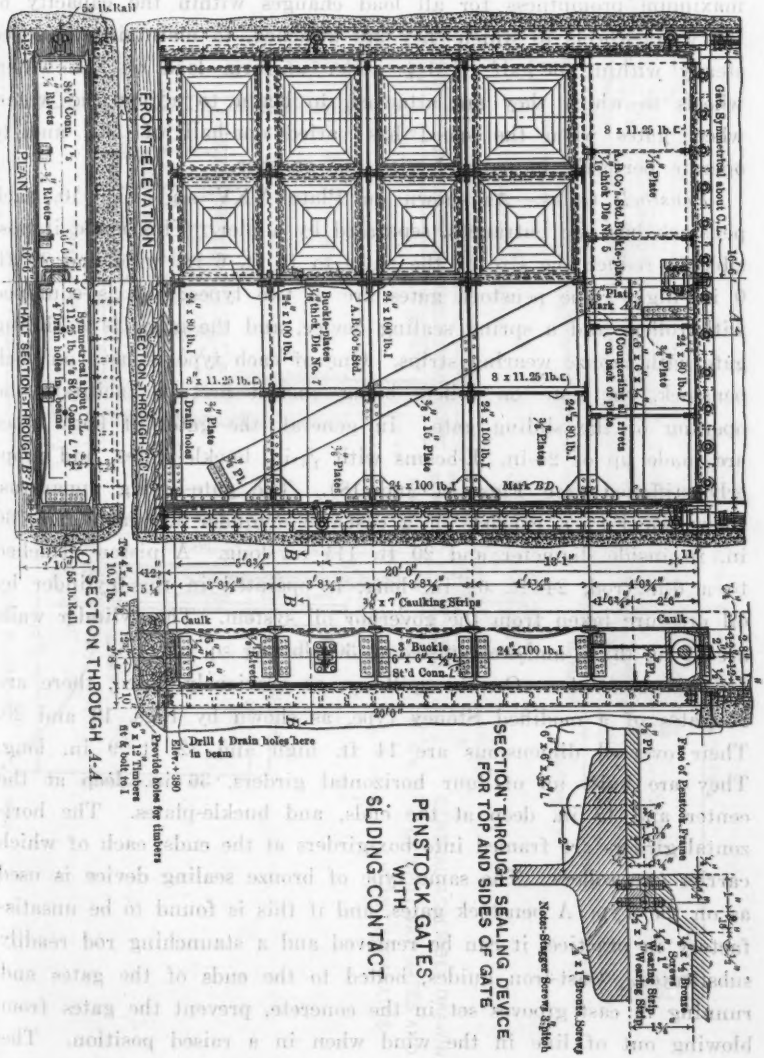

PENSTOCK GATES
WITH
SLIDING CONTACT
FRONT ELEVATION
SECTION THROUGH C-D
PLAN
HALF SECTION THROUGH B-B
SECTION THROUGH A-A
SECTION THROUGH SEALING DEVICE
FOR TOP AND SIDES OF GATE
55 lb. Rail
Caulk
Drill 4 Drain holes here in beam
Provide holes for timbers
Face of Penstock Frame
Note-Stagger Screws 3 inch

Fig. 18.

guaranteed to stand substantially steady when the speed does not vary, to be dead beat in action, and not to hunt, to correct with maximum promptness for all load changes within the capacity of the water-wheels to which they are attached, to maintain the speed steady within one-half of 1% under uniform load on the water-wheels to which they are attached, to begin to adjust the water-wheel gates when the speed has varied one-half of 1%, and to operate perfectly in parallel with one another.

Penstock Gates.—As shown on Plate XXV and Fig. 10, each penstock has two entrances, separated by a pier. This made it possible to reduce the size of the gates to 16 ft. 6 in. wide and 20 ft. 0 in. high. The penstock gates are of two types: one is equipped with rollers and a spring sealing device, and the other is a sliding gate with bronze wearing strips. One of each type is used on each penstock, the gate on rollers being opened first to facilitate the opening of the sliding gate. In general, the gates of both types are made up of 24-in. I-beams with $\frac{7}{16}$-in. buckle-plates, and properly stiffened (see Figs. 17 and 18). The gate-lifting apparatus, shown on Fig. 7, consists of a cast-iron cylinder for each gate, 30 in. in inside diameter and 20 ft. 11½ in. long. A piston attached to a 6-in. rod, 24 ft. 6½ in. long, is operated in this cylinder by oil pressure taken from the governor oil system. The cylinder walls are 1¾ in. thick, and were tested to 300 lb. per sq. in.

Spillway Gates.—On the spillway, as previously noted, there are 26 gates of a modified Stoney type, as shown by Figs. 19 and 20. Their over-all dimensions are 14 ft. high and 32 ft. 9 in. long. They are built up of four horizontal girders, 36 in. deep at the center and 18 in. deep at the ends, and buckle-plates. The horizontal girders are framed into box-girders at the ends, each of which carries two rollers. The same type of bronze sealing device is used as on the Type A penstock gates, and if this is found to be unsatisfactory in practice, it can be removed and a staunching rod readily substituted. Cast-iron guides, bolted to the ends of the gates and running in cast grooves set in the concrete, prevent the gates from blowing out of line in the wind when in a raised position. The gates are suspended at each end with flat steel ropes passing over drums at the top of the piers. The drums are actuated by shafting and gears, which may be operated from an electric hoist car mounted

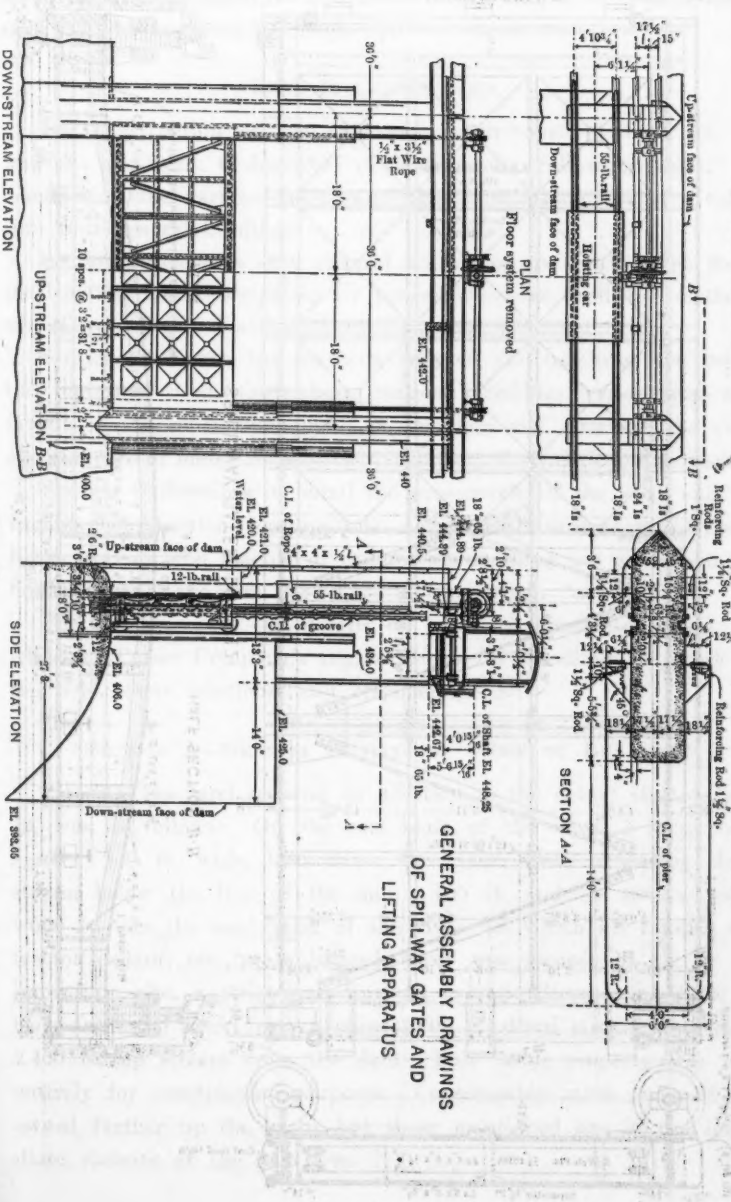

DOWN-STREAM ELEVATION
UP-STREAM ELEVATION B-B
PLAN
Floor system removed
SIDE ELEVATION
SECTION A-A
GENERAL ASSEMBLY DRAWINGS
OF SPILLWAY GATES AND
LIFTING APPARATUS

FIG. 19.

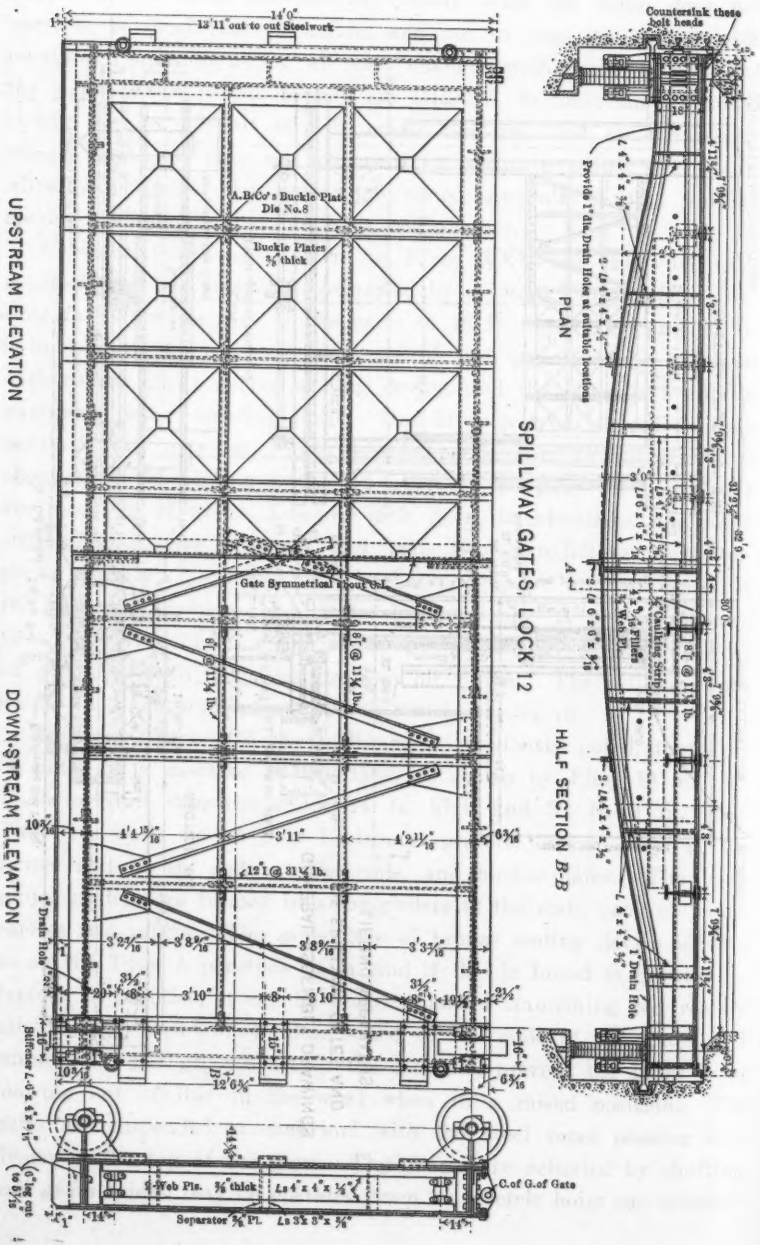

UP-STREAM ELEVATION
DOWN-STREAM ELEVATION
SPILLWAY GATES, LOCK 12
PLAN
HALF SECTION B-B
14'0"
13'11" out to out Steelwork
A.B.Co's Buckle Plate Die No.8
Buckle Plates 3/8" thick
Gate Symmetrical about C.L.
12 I @ 31¼ lb.
8 I @ 11¼ lb.
3'11"
4'2 11/16"
4'4 13/16"
3'10"
3'8 9/16"
3'3 7/16"
3'2 3/16"
1" Drain Hole
Countersink these bolt heads
Provide 1" dia. Drain Holes at suitable locations
2 - Ls 4" x 4" x ½"
7 - Ls 4" x 4" x ⅜"
2 - Ls 6" x 6" x ⅝"
Sheeting 3/8" Pl.
5/16" W.B. Pl.
C. of G. of Gate
2 Web Pls. ⅜" thick
Ls 4" x 4" x ½"
Separator ⅜" Pl.
Ls 3½ x 3" x ⅜"
Stiffener L 6" x 4" x 9.1#
(6" long cut to 6½")

Fig. 20.

on trucks above the piers. As added security, there is a duplicate operating car, equipped with a steam hoist, and, if necessary, both cars may be operated by hand.

Lock 12 Construction.

The construction of the dam and power-house at Lock 12, as laid out and built, is described in detail in the following pages. In considering the various features of this work, the paper is divided into four general headings.

Section A consists of a general outline of the work to be done, its location and description, its general features, and the methods which were planned and adopted for doing it.

Section B covers the methods adopted for handling the work, the materials used, the camps, railroad facilities, description and layout of the contractor's plant at the dam and quarries, the storage and care of materials, and the inspection of all work and materials.

Section C describes in detail the prosecution of the work and the various construction features—the coffer-dams, excavation methods, forms, mixing and placing of concrete, completing the work, and the final closure of the dam.

Section D covers the details of both the contractor's and the Alabama Power Company's organizations for handling the work in the field, their relations, and responsibilities.

Section A—General Outline of Work to Be Done.

Lands.—The land secured, in addition to the actual site of Lock 12, was as follows: On the east bank of the river, a strip averaging 1 800 ft. wide, back from the river bank, extending down stream below the line of the dam 1 400 ft. and up stream about 4 000 ft.; on the west bank of the river, on which all camps, contractor's plant, etc., were located, there was owned, in all, at the immediate site, a strip extending an average distance of 1 900 ft. back from the bank from a point 1 600 ft. down stream to a point 2 400 ft. up stream from the dam. This latter property was used entirely for construction purposes. Considerably more property was owned farther up the basin, but those mentioned are in the immediate vicinity of the dam site.

In addition to the lands described, at or near the site of the dam, on the west bank of the river, the timber rights to 3 250 000 ft. of lumber were purchased, and on the flooded lands purchased on the east bank of the river, there was another 1 000 000 ft., all of which was cut and used, furnishing nearly all the lumber needed for camps and other purposes.

Surveys at the Dam.—On August 5th, 1912, a party was put in the field at Lock 12 for the purpose of establishing the construction lines and elevations preparatory to starting the work. The tops of the 4-in. pipes, used by the U. S. Government for making shot borings on this site, were visible above the water surface, and the average line of these pipes was indicated as the up-stream face of the dam. This line was first permanently established, monumented, and thoroughly referenced, and an original stake, marked 1°, was established as Station 5 + 00 and so used on all surveys and drawings. This was the starting point of all measurements, and the stationing increased to the eastward from here. The up-stream face of the dam was considered as Station 0 + 00 for measurements at right angles, the stationing increasing down stream. In order that the main construction base line should fall inside the construction limits, a line parallel to and 10 ft. south of the face of the dam was run in, and all work was referred to this base line. Concrete monuments, with railroad spikes embedded in them and center-punched on line, were established on this 10-ft. line, at intervals of about 90 ft., for a distance of 1 000 ft. on each end back from the water's edge. On the west bank, to guard further against loss of lines, one of the monuments was carefully referenced. The original elevation left at this point by the Government was permanently established, and all work was based on it. All surveys of overflowed lands were based on this same datum, and deeds written were referred to it as their basis.

A diamond drill outfit was put on the work about August 5th, 1912, and a series of borings was made across the river, on the line of the up-stream face of the dam, in order to verify the original Government data.

Contract for Substructure and Dam.—A contract, under date of August 1st, 1912, was entered into for the construction of the substructures of the dam and power-house, and under it the work was

prosecuted to completion. It was of the cost-plus-a-fixed-fee type, in which, however, the Power Company, through the Engineering Department, had unlimited powers in the direction of the work, and exercised close supervision over all its details. The general requirements of the contract called for completion by December 31st, 1913.

The purchase of all materials was subject to the approval of the Power Company before orders were placed or any contracts made, and payments made under the contract were divided into three general items: (*a*), (*b*), and (*c*).

Item (*a*) covered the contractor's fixed fee;
Item (*b*) covered rental on plant owned by the contractor; and
Item (*c*) covered all expenditures made by the contractor in the prosecution of the work.

About one-half of the large plant was rented from the contractor, at a certain percentage per month, on an agreed valuation, and a list of the items of this plant was attached to and made a part of the contract. The selection of all plant used under Item (*b*) was subject to the approval of the Chief Engineer. Under Item (*c*) were made all expenditures in the prosecution of the work, and they were subject to approval by the Chief Engineer for all that were not the usual running items. The usual camp expenses were covered under a general approval. All plant actually purchased, the operation and maintenance of the railroad, and like items, were included under this item of the contract.

Sources of Materials, and Methods of Securing and Handling Stone, Gravel, and Sand.—Some time before the work was begun at the dam, investigations of all the surrounding country were inaugurated, for the purpose of finding suitable concrete materials for use in the dam. Prospecting parties were put in the field in the early part of August, 1912, to find sand and gravel pits, or stone quarries, as near the dam as possible. These investigations showed that all rock at the dam site, and for several miles up and down stream, would be unsuitable for concrete purposes, being a thin-bedded slate with numerous quartz seams running through it. No sand suitable for concrete was found at all. Gravel, in small quantities only, and in pockets, was found along the river, and such

sand as was found with it contained such high percentages of clay as to render it unfit for use without thorough washing. Early in the work, therefore, it became evident that sand would have to be secured from outside sources.

The only suitable stone in the district was found, after several months of prospecting, near Zion Church, in Sections 17 and 18, Township 23 N., Range 15 E. (see Fig. 3), and its area was more or less restricted. After carefully prospecting this site at Zion Church and another hill about a mile distant, known later as Ellison Quarry, both sites were purchased. All surface indications and drill borings indicated that Ellison Quarry would be the better of the two; later, however, the contrary turned out to be the case. Both sites were about 11 miles, *via* the construction railroad, from the dam, and were the only available sources of coarse aggregate and cyclopean stone found in the vicinity. Both these hills were heavily overburdened, and, after opening up Ellison Quarry and working it for several months, so many clay seams showed throughout the rock that it had to be abandoned. Although there was plenty of suitable rock available at Zion Quarry, it was only possible to work five derricks, and the stone could not be quarried with sufficient rapidity to satisfy the demands. For this reason, gravel and stone were secured from outside pits to fill out the supply. All the crushing operations were carried on at Zion Quarry, and a general view of the quarry and its layout is shown in Fig. 21.

A chemical analysis of the stone furnished from Zion and Ellison Quarries was as follows:

Aluminum and iron oxide	3.44 per cent.
Carbonate of lime	50.83 " "
Silica	9.00 " "
Carbonate of magnesia	36.38 " "
Combined water and organic matter	0.35 " "

The stone proved to be of excellent quality for concrete purposes, and was far superior to any purchased from outside commercial quarries.

All sand was furnished by contract at a price not to exceed 25 cents per cu. yd., f. o. b. cars, at Jackson's Lake, near Montgomery, Ala. There was a freight charge of approximately 70 cents per cu.

FIG. 21.—CRUSHER PLANT AT ZION QUARRY, SHOWING ALSO PORTION OF STORAGE BIN.

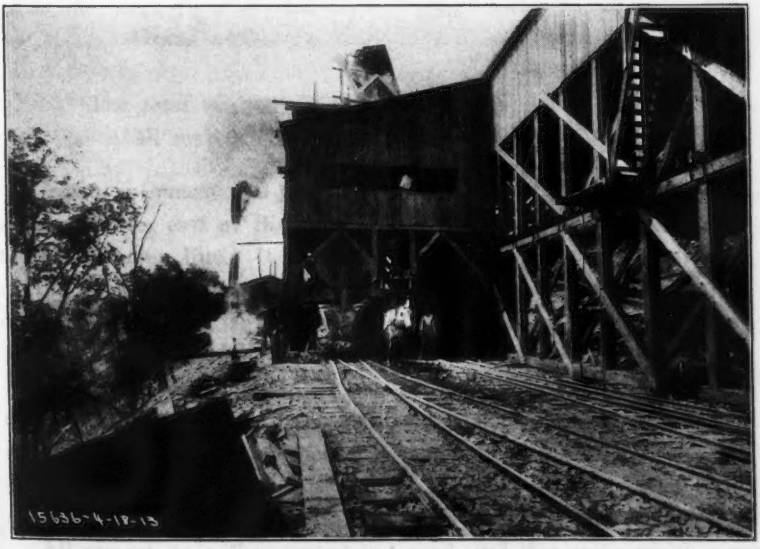
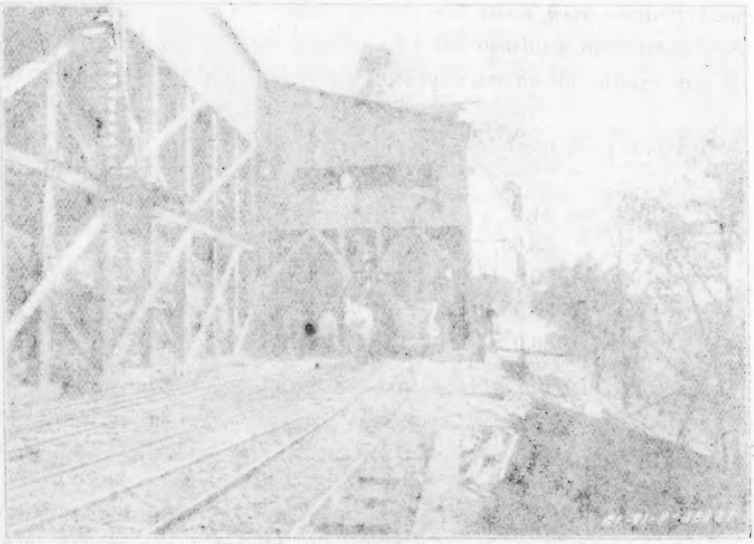


FIG. 22.—CONCRETE-MIXER PLANT.



yd. from Jackson's Lake to Ocampo, making the total cost per cubic yard at Ocampo, 95 cents. This sand proved satisfactory, although certain sections of the pits contained a good deal of loam, and it was necessary for the Power Company to maintain an inspector there constantly to supervise the loading and subject the sand to certain quick field tests for percentages of foreign matter.

All gravel secured from outside sources was washed, with the exception of that brought from the Company's pit at Elmore, Ala., which contained about 45% of gravel and 55% of sand. This pit was purchased to help out at the height of the concreting, and served its purpose.

The following tabulation shows the quantities of materials secured from various sources:

Crushed stone from Ellison Quarry....	11 031 cu. yd.
Crushed stone from Zion Quarry.....	115 207 " "
Cyclopean stone from Zion and Ellison.	7 670 " "
Stone purchased from outside sources.	13 998 " "
Washed gravel from outside sources...	31 031 " "
Elmore gravel (60% gravel).....	6 009 " "
Grand total.....	184 937 cu. yd.

The total quantity of sand used from all sources was..... 57 809 cu. yd.

Cement.—Cement was purchased from local mills, one in Alabama about 40 miles east of Birmingham and the other in Tennessee close to the Alabama line. It was tested at the mills by representatives of the Power Company, under the specifications contained in *Circular* No. 33 of the U. S. Bureau of Standards.

The cement was delivered to Ocampo in carloads of approximately 170 bbl., at a cost of $1.30 per bbl. net. At the dam site a weather-proof storage shed with a capacity of 15 000 bbl. was built and kept full until the work was nearly completed, when the storage was drawn on to tide over periods of car shortage caused by the cotton movement.

All cement, as well as gravel and sand, and that stone which came

from outside quarries, was contracted for by the Company and delivered to the contractor at Ocampo.

Lumber.—Rights to approximately 4 000 000 ft. of timber were acquired in the vicinity of the dam, and contracts were let locally to three small mills to cut it. All these mills were within a mile of the dam, and the sawed lumber was teamed by the Company to the point needed. It is to be noted here that, whenever possible, saw-mills should be on the construction railroad, or should have railroad service.

Timber proved to be very cheap in this section, the price of all the lumber used, of all classes, being about $10 per 1 000 ft. b. m. at the dam.

All the lumber for coffer-dams, camp buildings, bins, shops, stores, forms, etc., was supplied by the above timber and mills. Approximately, 6 600 000 ft. of lumber for all purposes—railroad, dam, and quarries—were used.

Commissary Supplies.—Practically all commissary and other running supplies were secured in the Montgomery or Birmingham market, being bought directly by the contractor. Before the construction railroad was ready for service, all these supplies were brought to Clanton, Ala., 13.6 miles from the dam, by road, and were teamed to the dam by local teams at 20 cents per cwt. Later, when the railroad was in service, a regular merchandise car was run by the Louisville and Nashville Railroad Company from Birmingham to Ocampo and thence taken to the dam on the Clear Creek Railroad. An arrangement was made with the Louisville and Nashville Railroad Company to run a regular package car exclusively for the work at the dam, and during the height of the work, there were two, and sometimes three, cars of merchandise daily to take care of the large force at work.

Coal.—Coal was contracted for by the contractor and delivered in hopper-bottom cars, *via* the Louisville and Nashville Railroad, at Ocampo, from which place it was carried on the construction railroad to the points desired. At the quarry and dam the coal was dumped through trestles directly in front of the boilers where it was to be used. On the railroad it was shoveled by hand out of the cars for engine uses directly into the tenders or to buckets moved by air hoists.

Coal was bought from various mines in Alabama, at a price of $1.50 per ton, at the mine, and a freight rate of $1.00, making the price, f. o. b. Ocampo, $2.50 per ton.

General Scheme of Handling Work at the Dam Site.—The general scheme and outline laid down and followed for the prosecution of the work contemplated the building of three coffer-dams and the handling of the work as follows:

1st.—Coffer-dam No. 1, which took in the west abutment, power-house, wasteway section, and 404 ft. of the dam, was built and the work within it excavated and concreted. This coffer-dam was designed and built to care for a flow of 75 000 sec-ft. in the river without flooding, and accomplished its purpose. Sufficient openings were left through the portion of the dam constructed within Coffer-dam No. 1 to care for a flow of 20 000 sec-ft. without flooding Coffer No. 2. The work within Coffer No. 1, it was anticipated, would be accomplished during the high-water stages of the river and that in Coffer No. 2 during the low-water, summer season.

2d.—The building of Coffer No. 2, turning the river through the culverts left in the first portion of the dam.

3d.—The construction of the second portion of the dam and east abutment, leaving more openings through it, with their bottoms at a higher elevation, in order to be able to pass a large flood if the river rose rapidly in the fall before the power-house end of the work was finished, and also making it possible for the reservoir to be drawn down if the water did rise to a considerable height before the work was completed.

4th.—The construction of the tail-race coffer-dam at about the same time as Coffer No. 2, and the excavation of the tail-race while the second section of the dam was being concreted. The power-house concreting was to be prosecuted while the first section of the dam was being built, but it was anticipated that it would be far behind that portion of the work, and would come to completion about the same time as the second section. Consequently, a small concrete wall, from the end of the power-house next to the wasteway section up stream, and a coffer-dam down stream to the tail-race coffer, were built in order that when the water was turned through the first section of the dam, the power-house and tail-race work would still be protected.

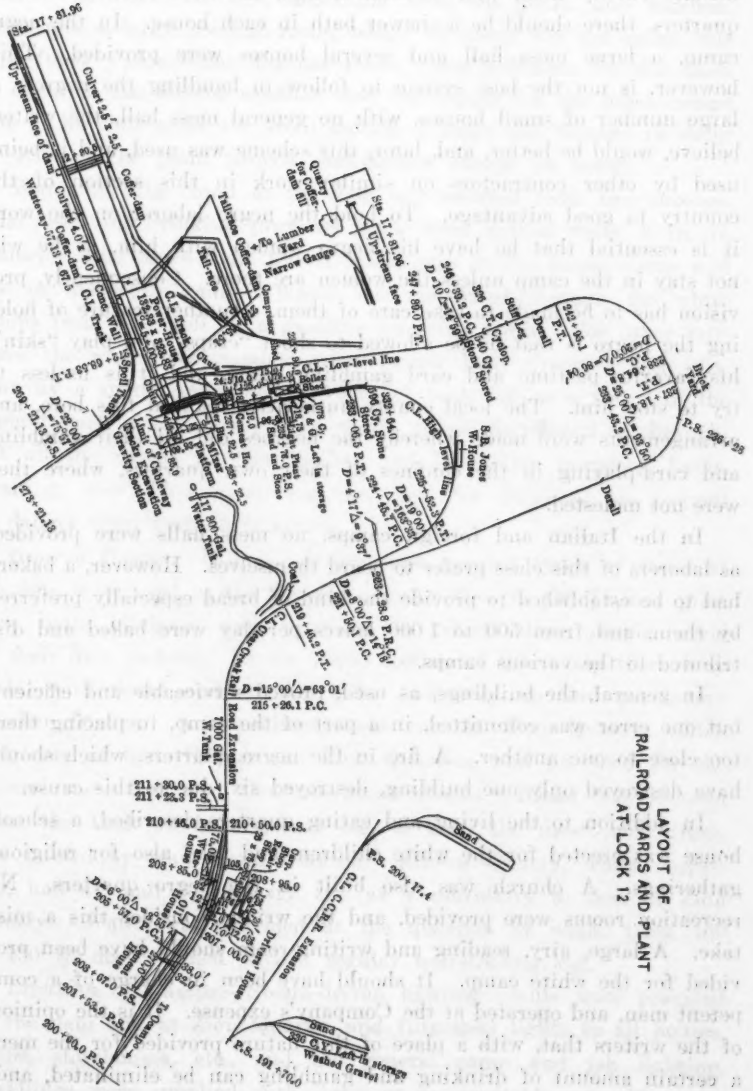
Section B—Methods Adopted for Handling the Work.

Camps.

The very first problem which presented itself in the prosecution of the work was the building of camps, and the providing of necessary messes and quarters for the men and houses for their families, as it was realized that unless there was proper food and comfortable habitations, it would not be possible to hold the men. It was anticipated that a very large force would be necessary in order to complete the work quickly and on time, and that, with such a large force, the sanitary and medical part of the work would need strict attention, and would be no small problem in itself. It was also essential that no contagious or infectious disease be allowed to gain a foothold and sweep through the camps, seriously hampering the work and giving the job a bad name. All camp buildings were constructed of wood, and were fairly well ventilated and cared for.

Lock 12 Camp.—At the dam five distinct sections of the camp were built. The white camp contained the commissary, white mess hall, bunk-houses, foremen's houses and families, and the houses of the contractor's staff, ice plant, bakery, storehouse, etc., in fact, this was the main camp. The engineers' camp was by itself, near the contractor's main camp No. 1, and contained a bunk-house, family dwellings of the staff, a mess hall, office, and a special house for the accommodation of Company guests and visitors, known as the "Guest House." The negro quarters were about ½ mile from the main camp, on one side of the railroad. About 400 yd. from the negro quarters was a fourth camp, for foreigners. The Swedish carpenters lived in a camp by themselves. The camps for the negroes and foreigners were combined into one large negro camp after the foreign laborers at the dam were discharged. Fig. 23 is a general map of the Lock 12 yards, railroad connections, etc.

About 1 000 persons could be accommodated at the main Lock 12 Camp, and at the height of the work it contained about that number. The topography at Lock 12 prevented any general scheme of laying out the buildings; they were placed on the tops of the ridges, with the drainage away from them toward the river and creeks. Where possible, a regular arrangement should be adhered to, and is also recommended from a sanitary standpoint. Some of the bunk-houses at this camp were too large, and did not afford proper ventilation at


Sta. 17 + 81.96
Upstream face of Dam
Waterway 51.5 x 67.5
Coffer-dam
Culvert 2.5' x 2.5'
Culvert 4.0' x 4.0'
Coffer-dam
Conc. Wall
C.L. Track
Power House 152.5' x 67.5'
Tailrace
Tailrace Coffer-dam
Compressor House
To Lumber Yard
Narrow Gauge
Quarry for Coffer-dam fill
Sta. 17 + 81.96 Upstream face
249 + 80.0 P.T.
D = 10° 00' Δ = 54° 00'
246 + 99.2 P.C.
250 C.Y. Concrete
240 C.Y. Stone Stored
Surface Drain
Derail
242 + 00.1 P.T.
C.L. Low-level line
Roller Shed
Derrick
Derrick
Derrick

night. A house of proper size would be one large enough to accommodate twenty men, well ventilated, and, in the skilled and white quarters, there should be a shower bath in each house. In the negro camp, a large mess hall and several houses were provided. This, however, is not the best system to follow in handling the negro. A large number of small houses, with no general mess hall, the writers believe, would be better, and, later, this scheme was used, and is being used by other contractors on similar work in this section of the country to good advantage. To hold the negro laborer on the work it is essential that he have his negro women with him, as he will not stay in the camp unless the women are there. Consequently, provision has to be made to take care of them. Another feature of holding the negro is that he be allowed to shoot "craps" and play "skin", his favorite pastime and card gambling game, and it is useless to try to stop him. The local county authorities realized this here, and arrangements were made whereby the negroes did all their gambling and card-playing in the confines of their own quarters, where they were not molested.

In the Italian and foreign camps, no mess halls were provided, as laborers of this class prefer to board themselves. However, a bakery had to be established to provide the kind of bread especially preferred by them, and from 500 to 1 000 loaves per day were baked and distributed to the various camps.

In general, the buildings, as used, proved serviceable and efficient, but one error was committed, in a part of the camp, in placing them too close to one another. A fire in the negro quarters, which should have destroyed only one building, destroyed six, due to this cause.

In addition to the living and eating quarters described, a schoolhouse was erected for the white children, and used also for religious gatherings. A church was also built in the negro quarters. No recreation rooms were provided, and the writers consider this a mistake. A large, airy, reading and writing-room should have been provided for the white camp. It should have been in charge of a competent man, and operated at the Company's expense. It is the opinion of the writers that, with a place of this nature provided for the men, a certain amount of drinking and gambling can be eliminated, and, furthermore, the superintendent can put his hands on some of the men quickly in times of emergency.

A small 1½-ton ice plant was installed at the rear of the white kitchen at Lock 12, and, adjoining both the plant and kitchen, there was a large cooling room, where all meat for the camp was kept fresh. This plant did not prove quite sufficient for the Lock 12 camp in the hottest weather, and the supply for the other camps had to be contracted for from outside sources and hauled in daily by team. The plant proved satisfactory except as noted.

Water for the camp at the dam was secured from two sources. That used for drinking purposes came from drilled wells, 6 in. in diameter, and more than 130 ft. deep, one in the negro camp and one in the white camp. Water was circulated through the white camps from this well by a gasoline pumping outfit. For other purposes the water was pumped from Yellow Leaf Creek, ½ mile distant, by a steam pump, to a 20 000-gal. tank set on a tower in the highest part of the camp, from which mains distributed it throughout all the camps for fire, boiler, and locomotive purposes, baths, washing, etc. This water was not used for drinking on account of danger from typhoid infection.

Quarry Camps.—Houses of the same general types as those at the dam were used at the quarries, except that, in the Italian quarters, little kitchens were provided in which these people could do their own cooking. In the negro section of this camp there were tents, with floors and wooden sides half way up and permanent wooden frames. A commissary was run at Zion Quarry, which took care of both Ellison and Zion Quarries, and provided for the general mess hall and the individual messes. Bread was secured from the Lock 12 bakery, and ice, to keep the meat, etc., fresh, was hauled by team from Clanton. Ellison Quarry, while operating, had a small commissary run as a subsidiary to that of Zion Quarry. Water was provided at the quarries by wells, from 100 to 150 ft. deep, drilled with a regular well-drilling rig.

Lighting of Camps.—Steam-driven lighting plants were provided at the dam and at Zion Quarry, and furnished lights to all houses, shops, storehouses, etc., and the general camp and job lighting. Dwellings were supplied with light, at an equitable rate, if so desired. Flaming arc lamps were used on the work proper and for general lighting throughout the camps. Later, when the transmis-

sion lines were connected up to Gadsden, the steam plant was shut down at Lock 12, and current was secured from this source.

Hospital, Medical Service, Sanitation.—All medical and hospital service was handled under a contract with a local physician who lived at the dam. Under the contract he was to render services at all the camps, provide all drugs, medicines, a hospital, nurses, etc., and receive in return 90% of the collections from the men, which amounted to $1 per month per man.

The doctor also acted as sanitary officer, but merely reported what he saw and had no authority to enforce his rulings, which was not a desirable feature. Sanitary precautions were taken by providing the pail system of closets and making the boxes of the closets fly-proof. The refuse matter was gathered nightly by a sanitary wagon, hauled a considerable distance from camp, and buried. All closets were kept well sprinkled with lime at all times. All kitchens and dining-rooms were screened, as well as practically all the family dwelling-houses.

The general health of all the camps was remarkably good. No contagious or infectious diseases originated at any of them. At the quarry camp several cases of typhoid fever developed among the Italians, and was traced to some bad meat which they had secured from an outside source. Only about six or eight cases came from this source, one of them being an inspector on the Resident Engineer's staff.

All accidents were treated immediately by the resident physicians, at the camps in which they occurred, and serious cases were brought to Lock 12 to the general hospital, where nurses were maintained.

Policing the Camps.—A regular force of deputies was maintained by the Power Company, regularly commissioned under the Sheriff of Chilton County, there being two deputies at Lock 12, one at the quarry, and one on the railroad. The most troublesome features arose from bootleggers bringing whiskey into the camps.

Camp Bosses.—Camp bosses looked after the cleaning and care of the camps; and their assistants, known as shack rousters, looked after the cleaning of the bunk-houses, etc. In the negro camp, the camp boss was a responsible white man who thoroughly understood the negroes and saw that they were not allowed to lie around

the camp, unless actually ill. He kept track of the various families occupying the small houses, looked after the checking of the dining-room, kept his eye on bad negroes, and, in general, maintained order. He also assigned quarters to new men coming in and looked after them, seeing that they got out to work properly.

Layout of Contractor's Plant and Details of Various Main Parts of the Plant.

Railroad Yards and Tracks.—At Lock 12 extensive yards and tracks for the storage and rapid handling of materials were provided; and a line was built down into the river bottom, below the dam, whereby trains and engines could get down on the cofferdam; this was known as the "Low-Level Line." There were two storage yards for sand and gravel where loading and unloading operations in no way interfered with regular train and switching movements. A long side track served the main cement warehouse, oil house, and general store-house, so that cars were unloaded directly into them. The shops were also served with another siding, but the arrangement used was a poor one, and detailed criticism of this feature of the layout is given elsewhere. The main line, known after it left the yards on its way to the dam as the "High-Level Line", served the mixer, cement warehouse, concrete material bins, and compressor plant, materials being unloaded directly to these various places from the cars. Fig. 23 is a map showing this detailed track layout.

Cableways.—Two traveling cableways spanned the river parallel to and immediately over the face of the dam. Together, the two cableways had a range of about 150 ft. up and down stream, that is, one could be placed over any portion of a strip 150 ft. wide at right angles to the line of the dam. The towers of these cableways were 95 ft. above the rail, their track elevation being about 440. The length of span, from center to center of towers, was 1 647 ft. The towers traveled on standard-gauge tracks and on standard M.C.B. car wheels and axles. The main cables were 2¼ in. in diameter and of patent, lock-laid, flat, steel strands on the outer covering. The cableways were also equipped with one button line, having eight buttons on it, one endless line, and one hoist line. The button and hoist lines were ¾-in., $\frac{6}{25}$-strand, Hercules, wire ropes.

The endless line was a $\frac{3}{4}$-in., $\frac{6}{25}$-strand, flat, wire rope. The button line was 1 700 ft. long, the hoist line, 2 100 ft., and the endless line, 2 600 ft. long. The cableways were operated by $12\frac{1}{2}$ by 15-in. engines, driven by steam, supplied from 80-h.p., locomotive-type boilers, all mounted on the trucks of the towers. The cableways had a rated capacity of 10 tons, and were operated under a sag of 80 ft. with a load of 6 tons.

In the early stages of the operation of the cableways, a good deal of trouble was experienced with carriers. The original carriers seemed to be too heavy, frequently breaking each other when they struck. Lighter carriers were made, and much of the trouble was eliminated.

Trouble was also caused by the breaking of two strands in No. 2 cable. The first breaks were welded by the oxy-acetylene process. The weld did not prove satisfactory, and they broke again; this cable was used for the remainder of the job with these strands cut out.

The buttons used on the button lines were of an obsolete type, and were not satisfactory; when one of them broke or slipped along the cable, it was necessary, in order to put on a new one, to lower the cable, cut out a section, and replace it, all of which was expensive and caused much delay. There is now manufactured a cast-steel button, made in two parts, which does away with the necessity for lowering the cable. A button of this type can be put on in an hour, and is a great improvement over the old one.

Extra hoist, button, and endless lines were kept in readiness at the head-towers, so that, in case of a breakdown at any time, they could be quickly and rapidly installed. The cableways handled practically 60% of the concrete, and were run both day and night. They also handled practically all the excavation in the early stages of the work. They were used for moving and erecting all derricks, etc., handling form lumber, iron, and materials of a like character, and were found to be exceedingly useful for the latter purpose.

Derricks.—All derricks used at the dam for excavating and placing concrete were of 5-ton capacity, of the wooden, traveling, stiff-leg type, with 37 to 40-ft. masts and 55 to 65-ft. booms. These derricks traveled on the up-stream and down-stream coffer-dams, parallel to the line of the dam, and on the bottom in the tail-race

and draft-tube excavation of the power-house. Later, when the concrete reached Elevation 380.0 in the power-house, three of these derricks were set up on high bents and were stationary.

All the derricks were equipped with $\frac{5}{8}$-in., $\frac{6}{19}$-strand, Hercules, steel-wire rope hoist and boom lines, and with 14-in. derrick irons and 12-ft. bull-wheels. Double blocks were used on the booms and single blocks on the hoist lines, and were operated by 7 by 10-in. engines, with swinging gears, driven by air. The traveling feature of these derricks seems to be an excellent one, as their increased radius of action is most advantageous. In this case the back trucks of these derricks traveled on the top of the coffer-dam, and the front trucks on a trestle resting on the bottom inside. Three of them were able to cover a length of 475 ft. of the dam, on the up-stream face. These derricks handled excavation in skips into cars run out on the up-stream coffer, and took concrete from the cableways, placing it in the forms, the cableways depositing a loaded bucket in a certain place and removing the empty bucket, the derrick then taking the bucket and dumping it in any portion of the form desired. They also took concrete from flat cars run out on the up-stream coffer-dam.

Mixer Plant.—The mixer plant was about 200 ft. south of the cableways, and discharged concrete to cars standing at Elevation 425. The plant consisted of two Model 64, 84-cu. ft. mixers, owned by the Power Company, and driven by an 18 by 24-in., horizontal, single-cylinder engine, rented from the contractor. This engine was run by steam supplied from the compressor plant, about 100 ft. away, as shown in Fig. 22. These mixers had a capacity of 20 batches per hour, their combined capacity being 3 200 cu. yd. for two 10-hour shifts. The highest run made was 2 220 cu. yd. Materials for concrete came in in hopper-bottom cars, and were dumped through a trestle into a bin having a capacity of 800 cu. yd. They were removed from this bin by a belt conveyor to a bucket elevator, and deposited in smaller bins over each mixer, from which they were drawn to the measuring hoppers. The belt conveyor and bucket elevator were driven by a 10 by 16-in., horizontal, single-cylinder, engine, also run by steam supplied from the compressor plant. The drive on both the mixers and elevators was with belts, to line shafts, and friction clutches. One of the drums of the mixers wore through,

comparatively quickly, due to the exceedingly tough and hard quality of the stone. This drum mixed approximately 70 000 cu. yd. of concrete. These drums were of ¼-in. metal, and the writers believe this to be somewhat too thin for this size. They suggest that a ⅜-in. shell would be preferable. Four sets of blades and one drum were worn out on one mixer, and three and one-half sets of blades on the other, in the course of the job.

Two elevators were installed during the progress of the work. The first was a No. 6, 18-in. bucket, stone elevator, 67-ft. centers, on the main pulleys. It did not have sufficient capacity to supply 1 000 cu. yd. per 10-hour day to the mixers, as required, and was taken out in August and replaced with a No. 8, 30-in. bucket elevator. A 32-in., 6-ply belt, 147 ft. long, was used, and had ninety-eight 30 by 17 by 10-in. steel buckets. After the installation of the larger elevator, there was no more trouble.

Water.—Water was supplied to the mixers through a 2-in. line from the main storage tank at the dam. Two small iron tanks with the necessary control valves, gauge glasses, etc., were equipped and set up overhead in the mixer-room, and all the water was drawn directly from these to the mixers.

Empty Sacks.—As soon as the cement was emptied into the hoppers, two laborers assigned to the duty gathered the sacks, shook them off to one side, packed them in bundles of 50, and stored them in the mixer cement house until from 20 000 to 30 000 sacks had accumulated, when a carload was shipped to the mills. The sweepings were sacked and used with the other cement.

Compressor Plant.—Practically all derricks, hoists, small pumps, air drills, small wood-boring tools, etc., were operated by air at a pressure of 85 lb., furnished by a central compressor plant which was near the mixing plant. The boilers of this plant also furnished steam for driving the mixing plant.

Two 24 by 30-in. straight-line air compressors, each with a capacity of 1 225 cu. ft. per min., furnished all the air. These machines were leased with other equipment from the contractor. The compressor and the mixer plant were supplied with steam by four 125-h.p. horizontal, locomotive-type boilers, leased from the contractor.

The compressors delivered air at 85 lb. pressure through a 6-in. main to a storage tank, 4 ft. 9 in. in diameter, 12 ft. long, and of

⅝-in. metal, located outside the building. A system of pipes was arranged to spray water constantly over this tank to keep it cool, and a special grade of oil was used in the compressors on account of the likelihood of explosion in the storage tank. The air was distributed to the principal points of use through a 6-in. main.

The construction of the compressor plant was commenced on November 16th, 1912, and the first compressor was turned over on January 14th, 1913. The plant was entirely completed and ready for business on February 1st, 1913.

Machine and Carpenter Shop.—The machine, carpenter, and blacksmith shops were all in one long building in the yards near the main storehouse. One line shaft, 84 ft. long, ran through the length of the building, and all machine tools were driven from it. The engine driving this line shaft was an 11 by 14-in., horizontal, single-cylinder engine owned by the Alabama Power Company. It was supplied with steam by a 30-h.p., vertical boiler, leased from the contractor. The engine and boiler-room was in the center and at the rear of the shops.

The machine shop contained the following tools:

One 30-in. radial drill press,
One 24-in. shaper,
One 18-in. by 10-ft. lathe,
One 1½-in. bolt threading machine,
One No. 94 Forbes, 2½ to 6-in., pipe cutting and threading machine,
One 16-in. by 5-ft. lathe,
One 30-in. diameter by 5-in. face power grindstone, and
One No. 4 Vulcan emery grinder.

The blacksmith shop contained the following:

One No. 2 Champion blower for forges,
Three blacksmith forges, home made,
Three anvils, from 250 to 300 lb.

The carpenter shop contained:

One No. 8 variety rip-saw and joiner,
One band saw,
One No. 402 bench grinder,

One No. 8 planer and matcher. This planer was very useful, and surfaced all form lumber; if the saw-mill had been operated by the Company, however, the planer should have been there.

The writers consider that the foregoing equipment served its purpose well, but there should have been added to the machine-shop a power hack-saw, a set of shears large enough to cut rounds up to 1 in. in diameter and flats up to 1 by 3 in., and a small steam-power hammer, on a job where there are a number of locomotives and cars to be taken care of.

The shops, as located, did not have sufficient floor and working space, and should have been separated from each other. Both should have been closer to the dam, and served by independent spurs from the railroad, so that cars of lumber could have been delivered to the platforms and yards direct. The laying-out floor of the carpenter shop should have been large enough to build one complete scroll case form on it, which would have taken a platform 75 ft. square. Also, there should have been a second laying-out floor, at least 50 by 100 ft., for ribs and other work of that kind.

The machine shop, if possible, should be as close to the work as convenient for all purposes, and should have at least two tracks for its own exclusive use: one for bringing machinery, etc., to be repaired at the shop; the other to serve as a general rip yard track. Where a railroad is operated and engines are taken care of, a pit for dropping wheels should be provided on one of these tracks.

Locomotive Cranes.—Two 10-ton locomotive cranes were leased from the contractor, and used in all features of the work. They did the work well, but the type used had only four wheels, which was hard on the track when traveling around. Two 1½-yd. clam-shell buckets were furnished with these cranes, and were used for unloading, storing, and reloading from storage all sand and gravel, and for coaling engines, etc. They were also used to assist in the disposal of excavation from the dam and tail-race, handling loaded skips. They were also used in unloading and erecting the contractor's plant and in reloading it when the work was completed. They were found exceedingly useful all through the work, but were not quite big enough for all purposes. After the work was started a larger crane was purchased, having a 50-ft., fixed boom and a 30-ft., trussed extension. This crane had a capacity of from 20 to 30 tons at 12 ft. radius, and

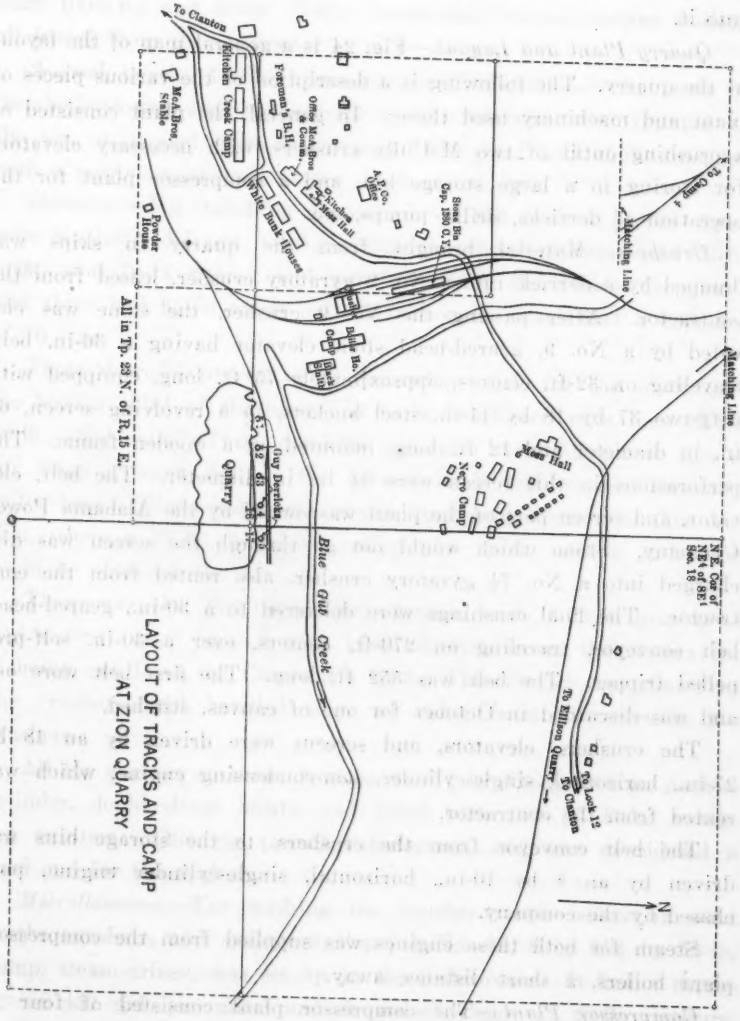

To Clanton
Foreman's D.H.
Office McA. Bros & Comm.
A.P. Co. Office
Stone Bin Cap. 1500 C.Y.
Matching Line
To Camp 4
Matching Line
Powder House
All in Tp. 23 N. of R. 15 E.
Quarry
Blue Gulf Creek
Mess Hall
Derricks
To Ellison Quarry
To Lock 12
To Clanton
N
LAYOUT OF TRACKS AND CAMP AT ZION QUARRY

Fig. 24.

was used at first for stripping at the quarry and loading cyclopean stone. Later, it was used in erecting the steel framework of the power-house and handling the heavy castings and machinery going into it.

Quarry Plant and Layout.—Fig. 24 is a general map of the layout at the quarry. The following is a description of the various pieces of plant and machinery used there: In general, the plant consisted of a crushing outfit of two McCully crushers, with necessary elevators for storing in a large storage bin, and a compressor plant for the operation of derricks, drills, pumps, etc.

Crushers.—Material brought from the quarry in skips was dumped by a derrick into a No. 9 gyratory crusher, leased from the contractor. After passing the No. 9 crusher, the stone was elevated by a No. 9, geared-head stone elevator having a 36-in. belt, traveling on 32-ft. centers, approximately 75 ft. long, equipped with fifty-two 37 by 18 by 11-in. steel buckets, to a revolving screen, 60 in. in diameter and 12 ft. long, mounted on a wooden frame. The perforations in this screen were 3½ in. in diameter. The belt, elevator, and screen part of the plant was owned by the Alabama Power Company. Stone which would not go through the screen was discharged into a No. 7½ gyratory crusher, also rented from the contractor. The final crushings were delivered to a 30-in., geared-head, belt conveyor, traveling on 270-ft. centers, over a 30-in. self-propelled tripper. The belt was 552 ft. long. The first belt wore out and was discarded in October for one of canvas, stitched.

The crushers, elevators, and screens were driven by an 18 by 27-in., horizontal, single-cylinder, non-condensing engine, which was rented from the contractor.

The belt conveyor from the crushers, to the storage bins was driven by an 8 by 10-in., horizontal, single-cylinder engine, purchased by the company.

Steam for both these engines was supplied from the compressor-plant boilers, a short distance away.

Compressor Plant.—The compressor plant consisted of four 18 by 24-in., straight-line compressors, having a capacity of 670 cu. ft. per min., and were rented from the contractor. This plant could operate a maximum of nine drills and six derricks. The plant was

not quite large enough for the needs on this work, and one more boiler and compressor would have been more satisfactory.

The compressors and other engines were supplied with steam by four 100-h.p., and three 75-h.p. horizontal, locomotive-type boilers, all leased from the contractor.

A coal trestle, 12 ft. high, was built immediately in front of the boiler-room, and hopper-bottom cars of coal were shunted to the top of it and dumped through. The coal was thus unloaded with a minimum expense at the point of use.

Derricks.—For handling stone out of the quarry proper there were six latticed-frame, steel-guy derricks, three of which had 75-ft. masts and 70-ft. booms, and were owned by the Power Company; and three had 65-ft. masts and 60-ft. booms. Each had a capacity of 10 tons.

These derricks proved to be somewhat weak for the work, and the bottom third of the mast had to be reinforced with four 30-ft., 60-lb., steel rails. These derricks were also very difficult to repair, for when a boom fell, or the derrick itself fell, it was generally badly bent. Timber derricks would have been better in this section, where a 70-ft., long-leaf, yellow pine stock, with a 12-in. top could be obtained for $25.

There was also one timber guy derrick, with a 73-ft. pine mast and a 68-ft. pine boom, using 14-in. derrick irons, the timber for it being cut from the Company's land. This derrick was placed at the crushers, and handled skips of stone from flat cars to the crushers.

All the engines on these derricks were 7 by 12-in., 24-h.p., double-cylinder, double-drum hoists, and fitted with size 3½ Dake swinging engines. This rig proved satisfactory, and was considered a good one for the work.

Miscellaneous.—For washing the crusher muck before it went to the crushers, and cyclopean stone, a No. 11, 6 by 10½ by 18-in. pump, steam-driven, was set up in the boiler-room. It had a capacity of 450 gal. per min., and furnished water, at 60-lb. pressure, through three 2-in. hose lines with 1-in. nozzles. The men stood on a washing platform and directed the water on the skip loads of stone just before they reached the crushers.

For pumping the quarry pit an 8 by 5 by 13-in. pump, having a capacity of 100 gal. per min., was found amply sufficient.

The heavy drilling in the quarry was done with No. 14, gasoline, portable, blast-hole, well drills. These were driven by a single-cylinder, gasoline engine, and drilled a 5⅝-in. hole, the whole outfit being mounted on the same frame, on trucks.

The lighter drilling was done with Ingersoll-Rand, E 24, rock drills, equipped with No. 44 air heads, No. 42 chucks, and No. 27 tripods, with weights.

A blacksmith shop with two forges was maintained at the quarry for the running work there, and also a No. 3 Leyner drill sharpener, which took care of the sharpening of all drill steel. It was operated by air, and was a great money and time saver. Where a great deal of drill work is done, and there is a large quantity of steels to keep sharpened, a tool of this kind should be used.

As a whole, the quarry plant was satisfactory and the layout good. The plant had a capacity of 2 000 cu. yd. of crushed stone per day, but the largest day's run was a little more than 1 000 cu. yd.

Storage and Care of Materials.

Two yards were provided for the storage of sand and gravel, and were served by spurs leading off the main line of the railroad west of the yards at Lock 12, as shown on Fig. 23. Each spur branched into a double track after leaving the main line, so that a locomotive crane operating on one track could unload a string of cars on the other without having them switched. Structural steel was also unloaded and stored in one of these yards. In addition, 1 600 cu. yd. of sand were dumped through a high trestle next to the compressor plant and about 300 ft. from the mixer bins. Run-of-pit gravel (containing about 55% sand) from Elmore was stored in the same piles with the sand. Washed gravel was stored in separate piles. At the quarry a storage of 8 000 cu. yd. of crushed stone was accumulated during the latter part of the job, when the concreting slacked up at the dam. This stone was stored beside the main line of the quarry branch, being dumped from 7-yd. cars over the side of a 10-ft. embankment. Storing this material at that place permitted the quarry to shut down, thus getting rid of a large overhead expense some time before the concreting at the

dam was finished. This material was afterward loaded by a locomotive crane operating a ½-yd. clam-shell bucket. A maximum quantity of 8 400 cu. yd. of sand was in storage on December 14th, 1913, and approximately 2 000 cu. yd. of Elmore gravel, and 2 800 cu. yd. of washed gravel. These quantities of materials were put into storage primarily in order to avoid hauling over the railroad any more material than was absolutely necessary after January 1st, 1914, the rainy season generally starting about then and giving a great deal of trouble from soft track, derailments, etc.

Cement Storage.—Cement was stored in two warehouses, the main one being in the Lock 12 yard, and having a capacity of 15 000 bbl. This building was absolutely damp-proof, and the cement stored there was well preserved. The other warehouse, accommodating about 2 000 bbl., was built at the mixers. This storage fluctuated considerably, as it was used simply to insure a supply of cement at the mixers at all times. Most of the cement used at the mixers was unloaded there directly from cars and was not kept in storage.

Sand.—So much loam and foreign matter was contained in certain sections of the sand pit that it became necessary to place an inspector there. He sampled the sand from fixed positions in the cars and made the ordinary rough test of mixing it with water in large test tubes and allowing the material to settle after being thoroughly shaken up. The coarse sand settling to the bottom was measured, and the percentage of clay or loam resting on its surface was determined. Cars containing more than an average of 10% of clay or loam, as determined by this method, were not accepted. This percentage was greater than is ordinarily allowed, but there were times when no cleaner sand could be obtained, and many carloads were rejected. The concrete obtained with the sand was very dense and of excellent quality. A very close watch was also kept on foreign crushed stone, as some of it was dirty; if it was bad, it was rejected, by an inspector at Ocampo, before being brought to the dam.

Watch was also kept and inspection made of incoming materials at the dam. The engineers in charge on the dam made regular trips during their respective shifts to the bins and the mixers, and kept close watch on the character of the materials running.

A good deal of trouble was caused by using washed sand from foreign pits by itself; being very coarse, it would not hold the cement

or water, and would not dump out of the buckets, thus causing considerable delay at times. The principal remedy was to dump cars of fine sand containing some loam alternately with the coarse sand, mixing it as much as possible in this manner. About 30% of this coarse sand was retained by a ¼-in. screen.

A series of tests was run on the Elmore gravel, and also on the washed gravel, to determine its fineness; 50% of the washed gravel would pass a ½-in. screen, and 55% of the Elmore run-of-pit gravel would pass a ⅛-in. screen.

The quantities of the various materials were determined both by measurement and by weight. All materials coming through Ocampo were weighed on a standard track scale installed by the Power Company. One hundred cars of material of each class were weighed and also measured, and the weight per cubic yard was thus determined. This factor was then used on all subsequent receipts. All quarry-crushed stone was measured in cars at the quarry by the inspector. All cyclopean stone was measured on cars by the yard clerk at Lock 12.

Section C—The Prosecution of the Work.

Railroad.—Owing to the site of the dam being 12 miles from the nearest railroad, one of the first problems to be solved was that of building a construction railroad. Time was very limited in which to construct this railroad to the site of the dam, as it was August when it was decided to start the work at Lock 12, and a railroad, under the contract, was to be completed to the dam by November 20th, 1912. As a matter of interest, plant was delivered there on November 26th. This short time allowance was practically the governing feature in the selection of the route adopted.

An old abandoned lumber road, known as the Clear Creek Railroad, extended from Ocampo, a point on the Louisville and Nashville Railroad 39 miles from Birmingham (Fig. 3), toward the dam, a distance of 17 miles. By extending this road 5 miles, the site of the dam was reached, making a total length of 22 miles. Surveys and investigations indicated that this would be the best route by which to get a railroad to the dam in at least 60 days. It was necessary to rebuild about 45 trestles on the old road and to re-tie it. The rails were still on the original ties, never having been taken up.

An inspection of Fig. 3 at once brings up the question: Why was the line not brought down from Ellison Quarry along Yellow Leaf Creek to the dam site, thus reducing the haul from the quarries materially? The answer is that, at the time Zion and Ellison Quarries were selected, the main line had been extended nearly to the dam, and about 80% of the work on the extension was done. If the quarries had not been at Zion and Ellison, the natural route for the line would have been as built.

In August the Power Company started a force of men from Ocampo to rebuild the old line, and a large grading outfit started to grade at the end of this line toward the dam. The work was completed, and steel was laid to the dam by November 26th, when carloads of plant were moved in. The roadbed, however, was still in a very bad condition, and 3 months more were spent with heavy forces re-tieing, surfacing, and opening up the old drainage channels which had become clogged. A steam shovel was put in, and the last 12 miles of the line on the dam end was ballasted, with shale taken out by the shovel and with crushed stone from the quarry. The first 10 miles of the road ran through a very rough hilly country, and the line was exceedingly crooked and full of heavy grades, though the roadbed was good and needed no ballast. On this section the maximum curve was 35° on a 2½% grade, the maximum grade being 4 per cent. On account of these physical conditions, Shay engines were used entirely on this division, five in all being purchased or rented.

The operation of the railroad was quite a problem in itself. In the height of the season more than 250 men were employed on it alone, and on some days more than 100 trains of all classes were operated. The road was divided into two distinct divisions, the first 10 miles being the Shay Division, using the Shay engines only, and the last 12 miles, including the Quarry Branch, operated by rod engines. During the period of greatest activity, outside of switching service, five Shay engines and six rod engines were used. Operating out of Ocampo, a 40-ton Shay engine could haul only 2 cars of sand or 3 cars of cement to a train. On the rod-engine division, 5- and 6-car trains were hauled with 60-ton engines.

On the rod-engine division, 8 miles from the dam, a branch line ran down to the quarries, a distance of 2 or 3 miles, and the heaviest movement was between this junction and the dam. Railroad head-

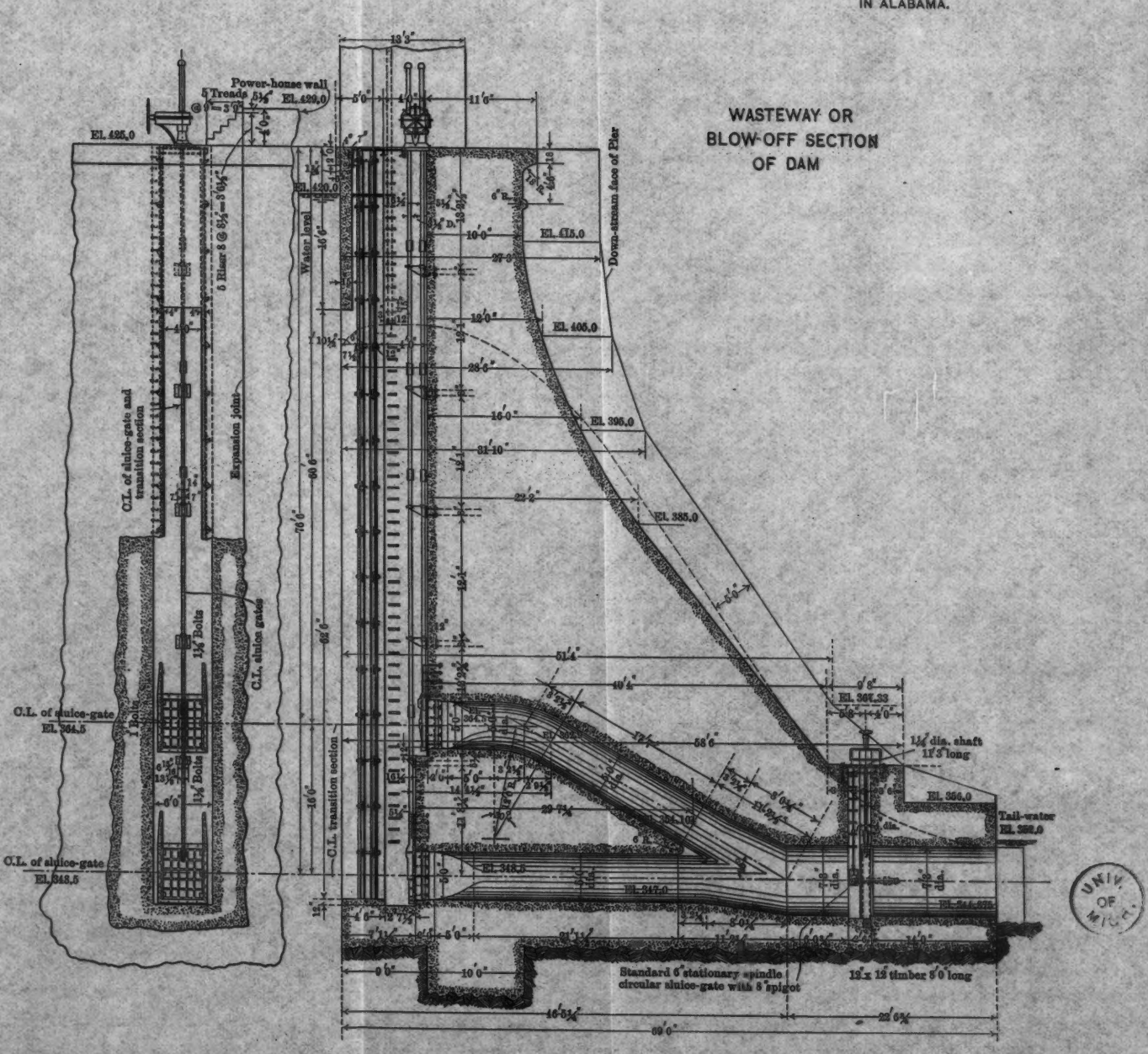
quarters were established at this point. A regular camp, similar in character to all the others, was built, shops were put up to take care of the rod engines, and a commissary was maintained. The superintendent, master mechanic, car repairers, and all rod division trainmen, hostlers, shopmen, etc., lived there. All Shay division trainmen, machinists, etc., lived at Ocampo, where there was a smaller camp; a well-equipped shop had to be maintained at this point, also, to take care of the Shay engines.

The Shay engines gave a great deal of trouble, their maintenance costs were high, and this division of the railroad was the most troublesome to operate. The use of Shay engines here was compulsory, as no other type could work on the grades and curves of the first 10 miles of the line. In view of the experience gained here, their use should be avoided, if possible. If a new line is built, it should be laid out for the use of rod engines throughout, and for heavy traffic. In the original road at this place 58¼-lb. steel rails were used, but were too light. The writers would recommend nothing lighter than 75-lb. steel where traffic is handled directly in the original cars from a connecting railroad, because, in such cases, net loads of 50 and 60 tons will have to be taken care of in modern cars of 100 000 lb. capacity. The tracks should be well ballasted and the roadbed should be standard in every respect.

The time element entered largely into the selection of the railroad as used, making it almost compulsory. Had there been more time, it is quite probable that an entirely new and independent line would have been built from some near point on the Louisville and Nashville Railroad to the dam. The building of such a line, however, would have delayed the work at the dam from 60 to 90 days, and it was not considered good policy to do this, with the short time available in which to comply with Government requirements as to the completion of the dam.

Stream Control.—The first essential feature of the work that had to be studied and carefully worked out in detail was that of the control of the river. All indications and records of the behavior of the Coosa for 15 years previous indicated that this in itself would be no small part of the problem, and that its solution would have a most important bearing on the progress of the work. All the Government records for a period of 15 years previous to 1913 indicated a

PLATE XXVI.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



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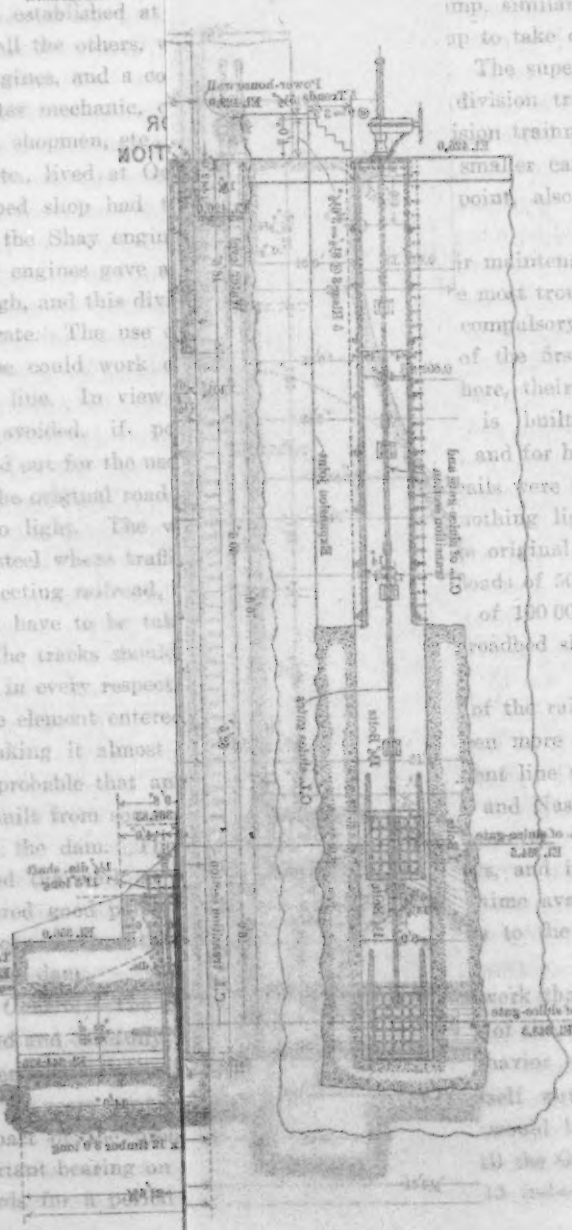


minimum flow of about 2 000 sec-ft. in the dry season and from 90 000 to 100 000 sec-ft. in the rainy season. Table 2 contains these records. It was necessary, therefore, to determine just how great a flood should be taken care of without flooding Coffer No. 1. It was not desired to build a coffer-dam to withstand the maximum flood, but, at the same time, it was considered necessary to build one of such height as to withstand an average high flood during high-water season. This was fixed at about 70 000 sec-ft., and calculations indicated that a dam, obstructing practically two-thirds of the stream, with its top at Elevation 360 would pass that quantity before flooding. The discharges, as determined by gauging the river below later, showed that with the river at this elevation above the dam, 85 000 sec-ft. were passing. It was anticipated that Coffer No. 1 would be flooded, and this happened three times during the high-water period. The first coffer was built as determined, and the dam was concreted. In the section of the dam concreted, however, three permanent culverts were left in the waste-way section, as shown on Plate XXVI, to fulfill a Government requirement calling for the passage of 4 200 sec-ft., with the water at Elevation 406. In the spillway section of the dam, there were left eleven tunnels, each 15 ft. wide and 16 ft. high, capable of passing 20 000 sec-ft. at Elevation 358, which was the elevation of the top of Coffer No. 2. From the top of each of these tunnels, two shafts ran upward through the spillway section and were used for filling them when the gates in front of them were closed. When the work inside Coffer No. 1 was above Elevation 360, this coffer was cut out and the river was turned through the eleven 15-ft. openings. Coffer No. 2 was then completed, and the work in it was finished. The work in Coffer No. 2, it was anticipated, would be accomplished during the low-water summer season, and the eleven openings would be ample to take care of any quantity up to 15 000 sec-ft., without flooding the coffer. The charts of previous years showed one or two flashy floods of from 15 000 to 25 000 sec-ft. during this average period. There was one flood of this character before the work inside the coffer was finished, water coming over into Coffer No. 2, delaying a portion of the work therein about a week, in October, 1913.

In the section of the dam constructed inside Coffer No. 2, ten 8 by 12-ft. openings were left, their bottom elevations being 357,

which was approximately 4 ft. below the top of the large openings left in Section No. 1. These openings were left as a positive safeguard to the power-house end of the work. This portion of the work was much slower than that on the dam, and it was feared that it would hang over until January, which was a very doubtful season, and a month in which a great deal of water might be expected. It was feared that, with only the eleven 15-ft. openings, a heavy and protracted flood would back up the river and there might be a rise sufficiently high to flood out completely the power-house work. The ten 8 by 12-ft. openings would have prevented this and, at the same time, helped to draw down the river more rapidly, if it rose to a considerable height.

Two types of gates were used to close these openings, a detailed description of which is given elsewhere. Figs. 15 and 16 give an idea of how these openings were arranged and closed.

Coffer-dams.—The construction of Coffer-dam No. 1 was begun on September 11th, 1912. It was composed of cribs 20 ft. square, roughly divided into two compartments; an 8-ft. compartment on the outside for clay and loam, and a 12-ft. compartment on the inside for rock. The cribs were built on the shore, launched, towed to their places, and sunk. After getting about 300 ft. out from shore, it was found necessary, on account of the increasing current, to stretch a ¾-in. wire rope across the river, by which the cribs were trolleyed out and placed. Before building a crib, soundings were taken, and the bottom timbers of the crib to be placed at that point were framed to conform as closely as possible to the shape of the bottom of the river. The coffer-dam was built of separate cribs to about Elevation 353. The joints in the timbers from that elevation to the top were then broken in the center of the cribs so as to make it as strong and as near one unit as possible. Sheeting, consisting of two thicknesses of 2-in. plank with lapped joints, was then placed on the outside of the cribs, and a toe-fill of sand, gravel, and loam, with a slope of about 1:1, was placed around the complete dam on the outside. Rock for filling and weighting the cribs of Coffer No. 1 was secured from a slate quarry about 200 ft. up stream from the dam, on the west bank of the river. Owing to the extreme swiftness of the current, the east or river end of Coffer No. 1, running up stream and down

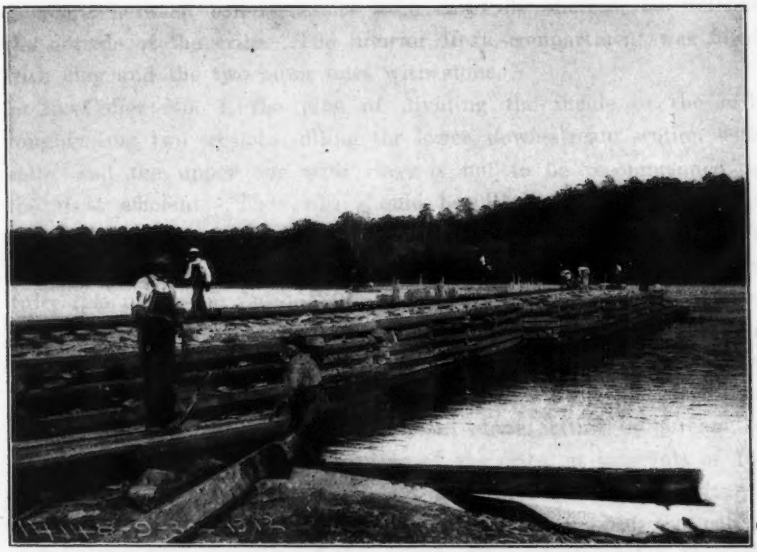
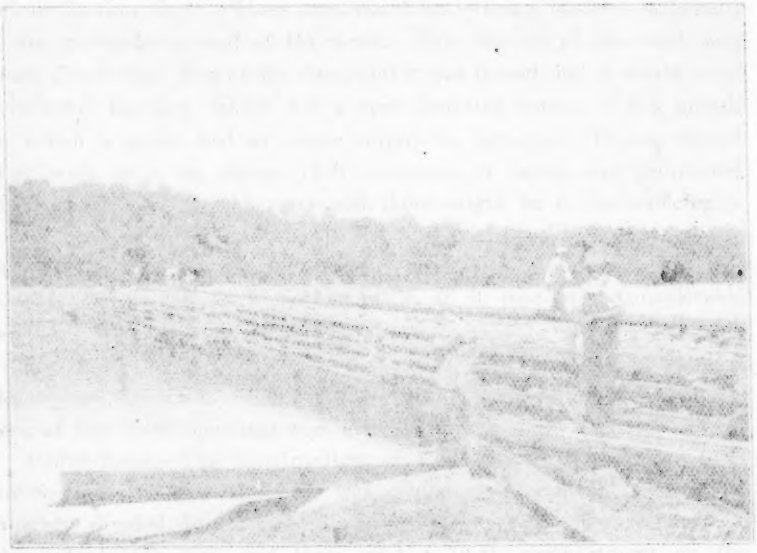
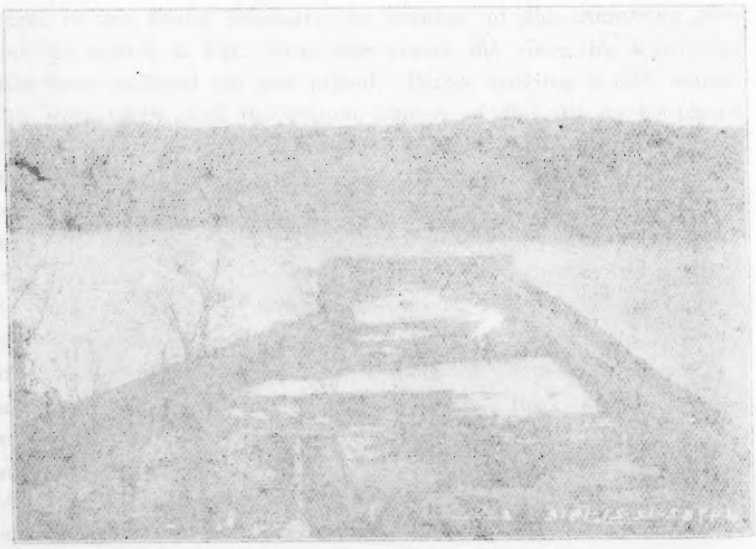


FIG. 25.—COFFER-DAM NO. 1, SHORTLY AFTER INAUGURATION OF WORK AT LOCK NO. 12.

FIG. 26.—COFFER-DAM NO. 1 COMPLETED. DRILLING OF FOUNDATION STARTED.





stream, was made 30 ft. wide with three 10-ft. compartments. All divisions between compartments were carefully sheeted, as well as the outside of the cribs. The interior 10-ft. compartment was filled with clay and the two outer ones with stone.

In Coffer No. 1, the plan of dividing the inside of the crib roughly into two sections, filling the lower, down-stream section with stone and the upper one with clay, is not to be recommended as the most efficient. The cribs should be filled entirely with stone, and all stoppage of leakage should be made with a substantial earth or clay toe-fill on the up-stream side, care being taken to sheet carefully this up-stream face. When the toe-fill has been properly made, if there is much current, a 2- or 3-ft. layer of rough rip-rapping should be dumped on the outside to protect it from wash. If there is much wash of the earth toe-fill while it is being placed, it can be stopped effectively by building small stone jetties up stream 30 or 40 ft., at right angles to the line of the cribs, at intervals of 150 ft. or more. This scheme was used here with excellent results.

In the case in question, the first flood that topped the coffers took out a large portion of this inside fill, and the space formerly occupied by the earth had to be refilled with stone. It was also found that this interior fill was not effective as a leak stopper unless it was made like the outer river end, that is, in a tightly sheeted interior chamber which absolutely prevented any wash through it.

The first timber used in making the coffer-dam was squared on four sides, but, later, timber squared on two sides was used, which decreased the time of handling and expense at the saw-mills. All coffer-dam timber was hauled from the mills to the dam—about ½ mile—by teams. On Figs. 25 and 26 is shown the beginning of construction of this coffer and its completion. Fig. 27 shows typical sections of the various coffer-dams.

Pumps were started on December 3d, 1912, but numerous leaks developed under the down-stream coffer, and the pumps were shut down until the toe-fill could be fully placed along the south section. A portion of the river bed was finally unwatered by December 13th, 1912. Coffer No. 1 was entirely completed by December 21st, its total length being 1 640 ft. It contained 525 000 ft. b.m. of timber, and took a little more than 3 months to complete. The

filling for this coffer, as well as for all others, was carried in 1½-yd. cars, which were filled by hand, from quarries on each bank of the river, with stone and earth from earth pits in the bottom, and hauled in trains of 4 cars each, by a mule, out on the coffer as far as it was completed. This method proved satisfactory, and no great trouble was experienced.

On May 20th, 1913, a coffer-dam, 10 ft. wide, was started below Coffer No. 1 to encircle the tail-race excavation (see Figs. 23 and 27). As the water was very low at this time, it was not necessary

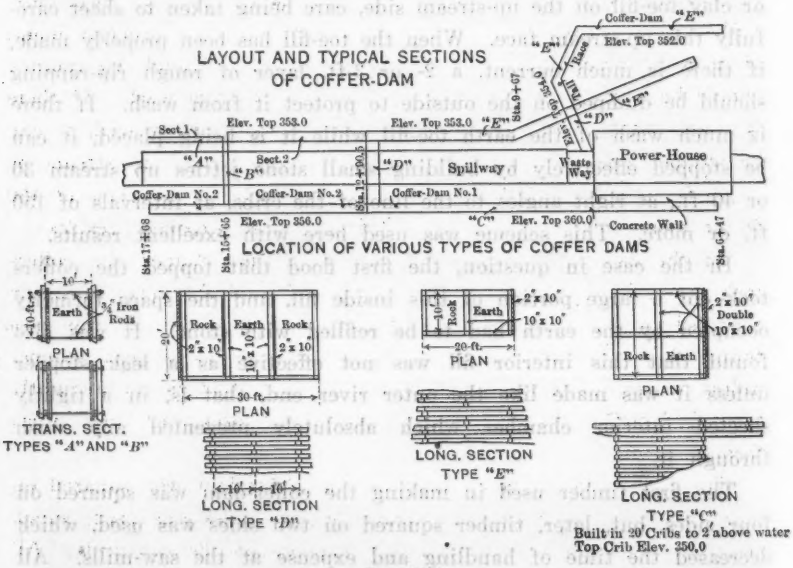


FIG. 27.

to build cribs on the shore, and they were constructed in place, the water being about 4 ft. deep. The average height of this coffer was 6 ft., the elevation of the top being 350.5 and its length 655 ft.

Coffer No. 2 was begun on June 17th, 1913, and the up-stream side was built of cribs, of the same size and in the same manner, as Coffer No. 1. A false deck was built above the water on this coffer and loaded with stone, allowing the water to run through the cribs until the up-stream section was practically completed and connection made with Coffer No. 1, when it was filled entirely with stone. This was done to keep the current as slow as possible to

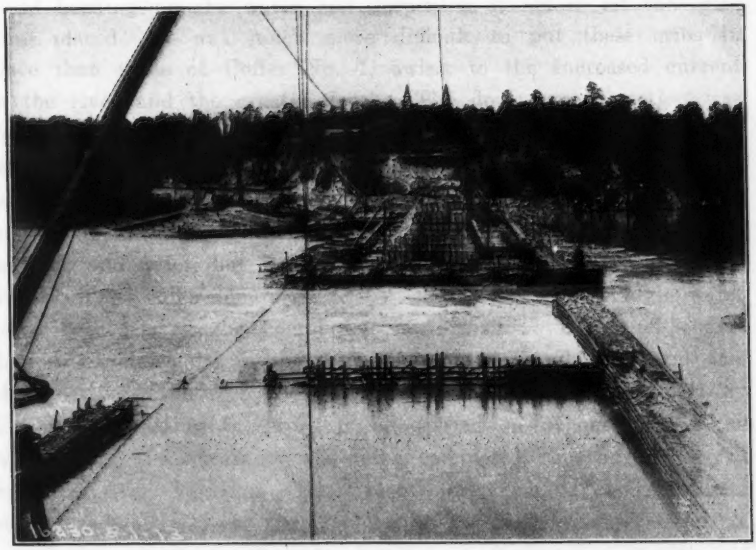
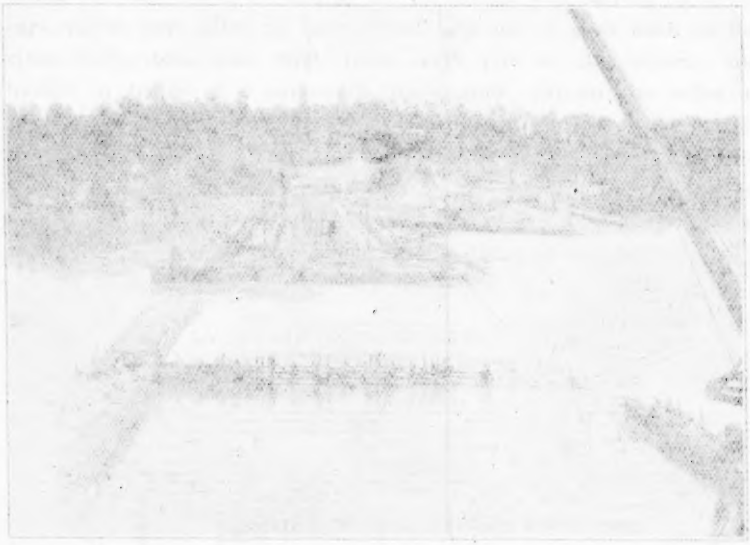


FIG. 28.—SECOND COFFER-DAM, SHOWING CONSTRUCTION OF LIGHT PARTITION RIBS UNDER WAY.

FIG. 29.—CUT-OFF TRENCH PARTLY FILLED WITH CONCRETE, SHOWING TYPE OF ROCK FOUNDATIONS.



avoid backing up the water and deepening it before all the cribs were placed. It was much more difficult to put these cribs in place than those of Coffer No. 1, owing to the increased current in the river and the greater depth. The down-stream section was built by using two 10 by 10-in. uprights with two 10 by 10-in. wales on the inside and 2-in. sheeting nailed on the inside of the wales, with only a loam fill. This makes a very cheap and quickly constructed coffer-dam, where there is not much head and the water is slack and quiet, but it is not very stable, and is easily washed away. This coffer-dam was partly constructed when high water came and delayed the completion and closure. A cut-off coffer-dam was constructed in the same manner as the down-stream one, about half-way between the shore and Coffer No. 1, as shown by Fig. 28, permitting the work to be carried on in one-half of the coffer, while the remainder was being completed, thus advancing the work by about 10 days. The average elevation of the up-stream side of Coffer No. 2 was 354, and that of the down-stream side, 351.5.

Closure of the north line of cribs of Coffer No. 2 with old Coffer No. 1 was made on August 20th, 1913. The closure in the south line was made about August 27th, 1913.

The total length of all the coffer-dams was 3 423 ft., and the total quantity of lumber and timber used was 898 984 ft. b.m.

Excavation in Dam and Power-House Foundations.—In general, the rock throughout all the foundation of the dam and power-house was a hard, blue, thin-bedded slate with numerous quartz seams running through it. Its outcrop crossed the axis of the dam at an angle of about 45° and dipped down stream, thus affording additional security from leakage and sliding. The general rough character of this bottom is shown by Fig. 29. Occasional soft streaks, from 6 to 8 ft. deep and from 25 to 75 ft. wide, were encountered, and these were worked entirely out, down to the hard blue slate. All rock excavation was drilled with Ingersoll, E 24 drills, operated by air. These drills made 2-in. holes, having a depth which varied according to the character of the rock. This feature was watched carefully by the engineers on the dam, and the depth of hole to be drilled in various sections was given to the drill foreman; the quantity of powder to be used was also specified. The quantity of dynamite varied from 1½ to 6 sticks of 40% Red Cross du Pont

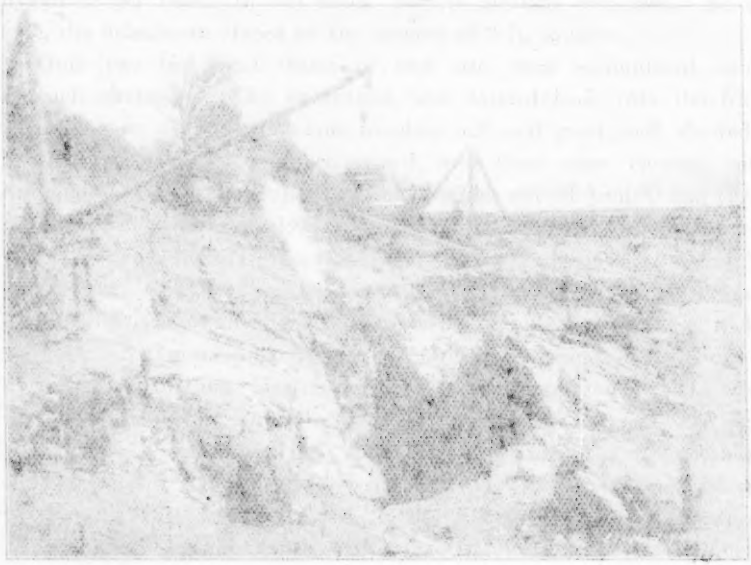
dynamite per hole. In wet holes, Forcite gelatine was used. As a rule, the holes were placed on the corners of 3-ft. squares.

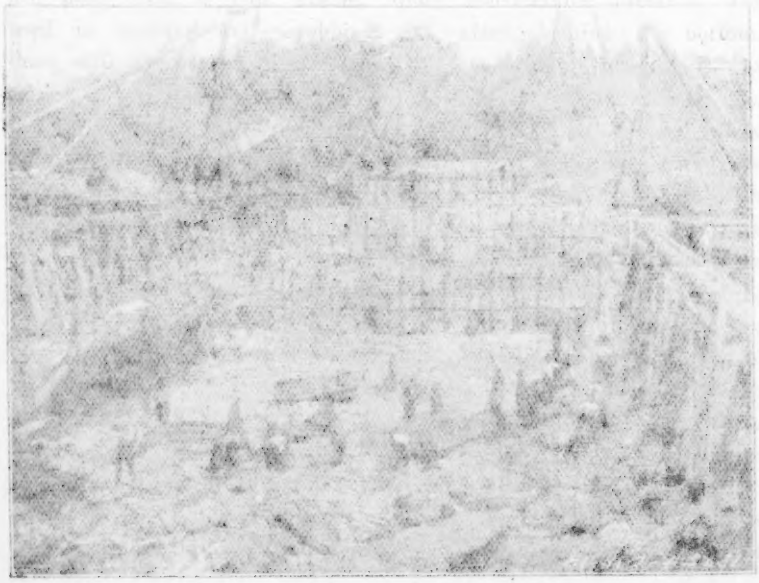
Only two bad mud seams of any size were encountered, one in each abutment. The excavation was carried back into the hill in each case until these seams pinched out and good rock showed. Some small seams were encountered, and these were cleaned out thoroughly and grouted under pressure. The cut-off trench was carried from 6 to 15 ft. below the general level of the excavated bottom; its excavation is described under the heading "Channeling." No differences in the character of the rock in the "cut-off" trench from that in the bottom, already described, were found, except that the rock was somewhat more compact in the bottom of the cut-off trench.

After the drilling, blasting, and excavation of the materials was completed, and the bottom was fairly clean and compact looking, a gang of 20 men was put in on a 75-ft. section and generally worked there 2 days, barring, wedging, and picking the surface of the work. This was continued until all loose slabs of slate were removed, and then scrubbing, washing, and cleaning were continued until the foundation was absolutely clean and free from all loose materials. Wire brooms were used all over the bottom, and a 2-in. fire hose, with a ½-in. nozzle, under considerable pressure, was used in washing and scrubbing. If, after cleaning the bottom, there still appeared to be pockets or layers of soft material, it was again drilled, shot, and excavated, and then finally cleaned up. Wherever leaks of any consequence occurred (which were generally low down in the "cut-off" trench), a cone of spalls and mortar was built, completely enclosing the leak, the water being led out of a 1½-in. pipe from the bottom of the cone and another pipe rising out of its top. When the cone had set, the drain from the side was plugged and the water was allowed to rise in the vertical pipe. This pipe was brought up until some 15 or 20 ft. of concrete lay over the leak, and then it was grouted under pressure up to 50 lb. Where several bad leaks were encountered, a well was formed in a suitable place in the "cut-off" trench, and all the surrounding leaks were led to it through 1½ and 2-in. pipes. A pump was set up nearby to keep the water lowered in this well until the concrete was well set. The well was brought up to such height as was necessary to bring the water level to a stand, then it was pumped

FIG. 30.—DEEP GRAVEL POCKET ENCOUNTERED IN FOUNDATION, BETWEEN UNITS NOS. 1 AND 2 OF POWER-HOUSE.

FIG. 31.—GENERAL VIEW, SHOWING METHOD OF HANDLING WORK AND TYPE OF WALL FORMS USED.





down, the forms and dirt were cleaned out of it, and grout pipes were brought up from the drains leading into the wall, and then filled with concrete. After the concrete had set, the leaks were grouted back, under pressure, through the original drain pipes. There were no serious leaks in any portion of the work.

The rock under the power-house was slightly heavier and more dense than in other portions of the foundations, but at the west end, between Units 1 and 2, and partly under Unit 1, a large cavity filled with sand, gravel, and slate boulders was found. This cavity extended the full width of the power-house up and down stream, and was about 50 ft. wide on top, narrowing to about 15 ft. at the bottom. It was wedge-shaped, with the small end down stream. Fig. 30 is a view of this hole. The lowest part of the hole was at Elevation 291.0, or 35 ft. below the bottom of the draft-tubes. The hole itself, the writers believe, was of the nature of an old "geological mill," and was worn out by the whirling around of gravel and sand in pot-holes, these pot-holes wearing eventually into each other. The hole was cleaned to the bottom, as a dentist cleans a cavity in a tooth, and then filled with concrete. The hole extended both up and down stream beyond the limits of the excavation, and may have been of considerable length. Two derricks were used in excavating it, handling 1½-yd. cars from the hole to a narrow-gauge track, the material excavated being used for a toe-fill on the outside of the coffer-dam. After solid rock was reached at the bottom, 16-ft. holes were drilled in various places to make sure that there were no caverns below, but the rock drilled solid and clean. The 2 200 cu. yd. of concrete necessary to fill this hole to the general level of the surrounding bottom was poured through a chute, which was built directly from the mixer.

The following quantities were excavated:

Dam	Earth	3 379 cu. yd.
	Rock	15 442 " "
Power-house	Earth	8 572 " "
	Rock	11 818 " "
Tail-race	Earth	8 029 " "
	Rock	11 245 " "
Total		54 845 cu. yd.

Excavation was started on the dam and power-house foundations on December 17th, 1912, and completed on October 20th, 1913.

Channeling the Cut-Off Trench.—A cut-off trench, 10 ft. wide, from 6 to 15 ft. below the general bottom of the dam, extending under both the dam and power-house, and well keyed into both banks of the river, was cut. Two channelers were used for both sides of a large portion of this trench.

Time was lost at the beginning of the work in starting these channelers, and it was seen that they would have to be supplemented in order that the excavation of the trench might keep up with that of the general bottom, and be ahead of the concreting. Two quarry bars were purchased and two Ingersoll, E 24 drills were set up on each, channeling in the power-house foundations. These drills put down holes 6 in. apart on the line of the trench. This method was not satisfactory, as the rock was badly shattered in blasting and the trench had to be carried much deeper than was necessary in order to get below the shattered rock. The channelers gave a clean cut and much better job all around, and are far superior to the drill work.

In the abutments only the up-stream face of the abutment was channeled by drills, and no actual cut-off trench was made, the whole excavation into the hill being practically such a trench. The thickness of the channeler steel varied from 1¼ to 2½ in.

Throughout the work on the cut-off trench, 40% Red Cross dynamite was used, with the exception of a few wet holes where 40% Forcite was used.

Disposal of Waste Materials.—At the beginning of the work, before the derricks were in position to handle the excavation, all excavated material was moved by cableways, in 4-yd. steel skips, to 7-yd. dump cars hauled by an engine on the Low-Level Line, the material being used to widen the embankment for that line. This method, however, was very slow and unsatisfactory, it being hard to stop and dump a heavy skip over a 7-yd. car. Later, the skips were landed on flat-cars, taken about 1 000 ft. up the Low-Level Line, on a steep hillside, where a large guy derrick was erected to pick them up and unload them. A large part of the draft-tube excavation in the power-house was made in this way. When the traveling derricks were put in service on the up-stream coffer-dam, a track was laid on the coffer-dam and a dinky locomotive, handling 7-yd. cars, took the excavation from

these derricks. It was lifted out of the bottom in 4-yd., steel skips and dumped on a slanting platform built for the purpose. The material was then dumped over the side of a 10-ft. bank into the river about 200 ft. up stream from the dam. Later, these derricks landed loaded skips on flat cars, handled by a dinky engine and a locomotive crane, on the dump already mentioned, picked them off and wasted them down the bank into the river. Material in the excavation, out of reach of the up-stream derricks, was picked up by those traveling on the down-stream coffer-dam and passed across to the up-stream derricks which, in turn, disposed of it as described.

The tail-race excavation was handled and disposed of in a different way: Two traveling derricks started at the east side of the excavation and backed off toward the west as they cleaned up the excavation. The bottom was drilled, shot, and then placed by hand in 4-yd. skips. These were picked up by the derricks and discharged into 7-yd. dump-cars, which were hauled out of the bottom by a cable operated by a regular hoisting engine, and disposed of down stream about 500 ft. from the side, on an embankment. All the tail-race excavation was made in this manner very quickly and satisfactorily.

In general, the following criticism is offered of the methods used on this work: A cableway is not satisfactory in handling the excavated materials, as it is not only expensive, but slow. A better scheme for the work in Coffer-dam No. 1 would have been a system of standard-gauge tracks laid down from the west bank of the river, through the tail-race, to its extreme east end. The excavation should then have started at that end, backing off toward the west, the spoil, in skips, being lifted with the derricks and discharged into 7-yd. dump-cars, hauled by dinky engines. The power-house excavation could have progressed rapidly, also, at the same time, and both cableways would have been free to erect derricks, get plant and timber out to the job, and handle concrete, thereby advancing the work greatly. The writers are of the opinion that, as a rule, the cableway should be restricted to moving plant, derricks, forms, and general work, and also such concreting as is advantageous to place with it, and that it should not be made the main method for the transportation of all materials on the job, as it was here.

Forms Used for the Dam.—The forms used on the vertical faces of the dam were of the cantilever type, built in sections, 12 ft. long

and 6 ft. high, with upright posts extending 6 ft. below the bottom of the sheeting. Figs. 29 and 31 give a general idea of these forms. All the sheeting was sized and dressed to 1¼ in. in thickness, and all vertical posts were made of two 4 by 8-in. timbers nailed together, with 1-in. blocks between them at the top, middle, and bottom. The waling pieces were 4 by 8-in., or 6 by 8-in., laid flat. These forms were built complete and raised in one piece, from one lift to another, by a 3-ton chain block hung from an **A**-frame, guyed back with form wire to hook-bolts in the concrete. This particular kind of form was not used throughout the job, for, as the work progressed and carpenters became rather scarce, sections of this type were abandoned, and 3 by 6-ft. panels were made in the carpenter shop and simply tacked to the posts. This type was practically the same as the first and was handled more quickly and easily than the forms of heavier section.

The following method was used to hold the forms in place: When a form was completed, ¾-in. bolts, 22 in. long, with an ogee washer on one end, were placed in them, 10 in. below the top, the bolts having a 10-in. hold in the concrete when it was poured around them. Also, when the pour in the form was completed, and while the concrete was still soft, ¾-in. bolts, 12 in. long, with a hook on one end, were set in the concrete opposite each post and about 6 ft. back from the face of the form. When a section form or panel form was raised, it was lifted until the bottom of the sheeting was above the bolts set 10 in. below the top of the previous pour of concrete. The posts were fitted on these bolts which held the form in place, passing through the slot between the two 4 by 8-in. posts. A waling piece at the top of each section was used to wire the form back to the hook-bolts placed in the concrete. Wedges were placed at the bottom of the post to line up the form. The panel forms were handled in the same manner, the loose posts being placed on the bolts previously set in the concrete, and the panels dropped into place behind them and tacked to the posts. As the speed of filling the forms increased, two walings were used instead of one. All expansion joint forms were held in this same way. For the curved section of the dam, on the down-stream face, ribs were cut to the proper radius in the form shop and brought down to the job where sheeting was nailed on them in place at the dam. The ribs for all curved forms were made of two 2 by 8-in. timbers nailed together, with a 1-in. spacing block between them, leaving a 1-in.

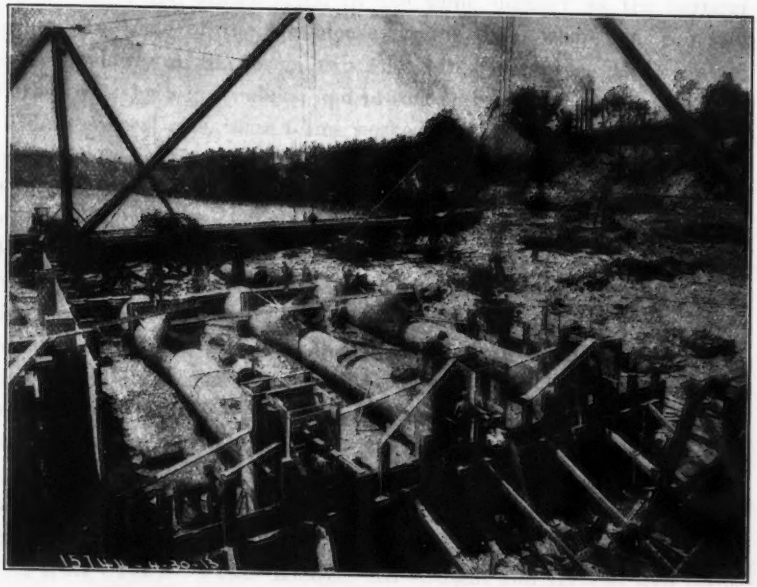


FIG. 32.—FORMS FOR WASTEWAY OR BLOW-OFF CULVERTS.

FIG. 33.—SCROLL CASING FORMS IN PLACE DURING CONCRETING OF FIRST STAGE OF CONSTRUCTION.

slot to catch the anchor-bolts, in the same manner as the vertical forms. In some of the big blocks on the dam, where the concrete came up slowly on the forms, two 2 by 8-in. pieces were used in place of the two 4 by 8-in. timbers, and proved satisfactory.

As a general rule, these forms worked well and were fairly economical of timber. However, it is not the fastest method; after a form is poured, sufficient time has to be allowed for the concrete to set properly and for the bolts to take hold. More rapid work could be done with a form of the same general type, but built up continuously, releasing the bottom sections as the opportunity arose. In building the forms continuously, one could be raised as soon as the carpenters could walk on the concrete, and the next pour could be made from 12 to 24 hours sooner than with the type used. The latter, however, would take considerably more timber, but the speed would be increased, and in a country where timber is very cheap and speed a big consideration, the continuous type might pay. Fig. 32 shows the type of forms used in moulding the wasteway culverts in the section of the dam adjoining the power-house.

Forms Used for Power-House.—All straight and vertical wall forms in the power-house were of the same type, and were handled in exactly the same manner as the dam forms.

All special or curved forms, such as for draft-tubes, scroll casings, and penstocks, were made in the form shop according to detailed drawings furnished by the Company's engineers.

The type of scroll casing used (Fig. 10), was more or less of an innovation, and there was considerable discussion at the beginning of the work as to the practicability of moulding such a complicated opening. The form work, as can be readily seen from Figs. 34, 35, and 36, was highly complicated, and, even when carefully detailed, was difficult of execution in the field, for several reasons: First, the average carpenter foreman is out of his depth on a complicated drawing of this kind; secondly, the setting and lining up of a large form of this character is difficult of execution; thirdly, unless the laying out of the forms is done carefully, they are hard to get out. In the case at Lock 12, the engineers made all the calculations and furnished a sufficient number of detailed drawings to enable the work to be laid out and assembled in the form shop. A competent engineer assisted the carpenter foreman in reading these drawings properly, and laying

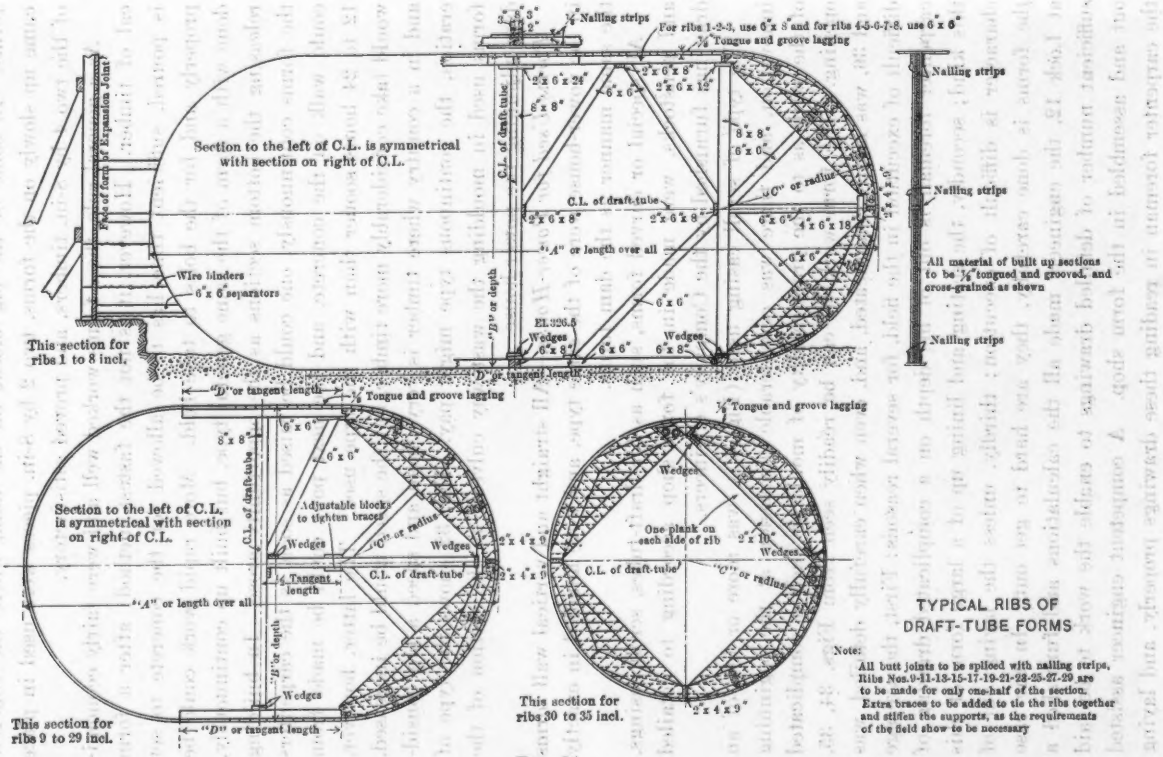

Nailing strips
For ribs 1-2-3, use 6" x 8" and for ribs 4-5-6-7-8, use 6" x 6"
Tongue and groove lagging
Face of form at Expansion Joint
Section to the left of C.L. is symmetrical with section on right of C.L.
C.L. of draft-tube
"O" or radius
"A" or length over all
"B" or depth
Wire binders
6" x 6" separators
This section for ribs 1 to 8 incl.
EL 326.5
Wedges
"D" or tangent length
Nailing strips
All material of built up sections to be ¾" tongued and grooved, and cross-grained as shown
Adjustable blocks to tighten braces
Tangent length
This section for ribs 9 to 29 incl.
One plank on each side of rib
This section for ribs 30 to 35 incl.
TYPICAL RIBS OF DRAFT-TUBE FORMS
Note:
All butt joints to be spliced with nailing strips. Ribs Nos. 9-11-13-15-17-19-21-23-25-27-29 are to be made for only one-half of the section. Extra braces to be added to tie the ribs together and stiffen the supports, as the requirements of the field show to be necessary

FIG. 34.

out the various parts of the form. The greatest care was exercised in setting up the forms and holding them in place. The work was handled in this manner with no great difficulty, and the final results were excellent.

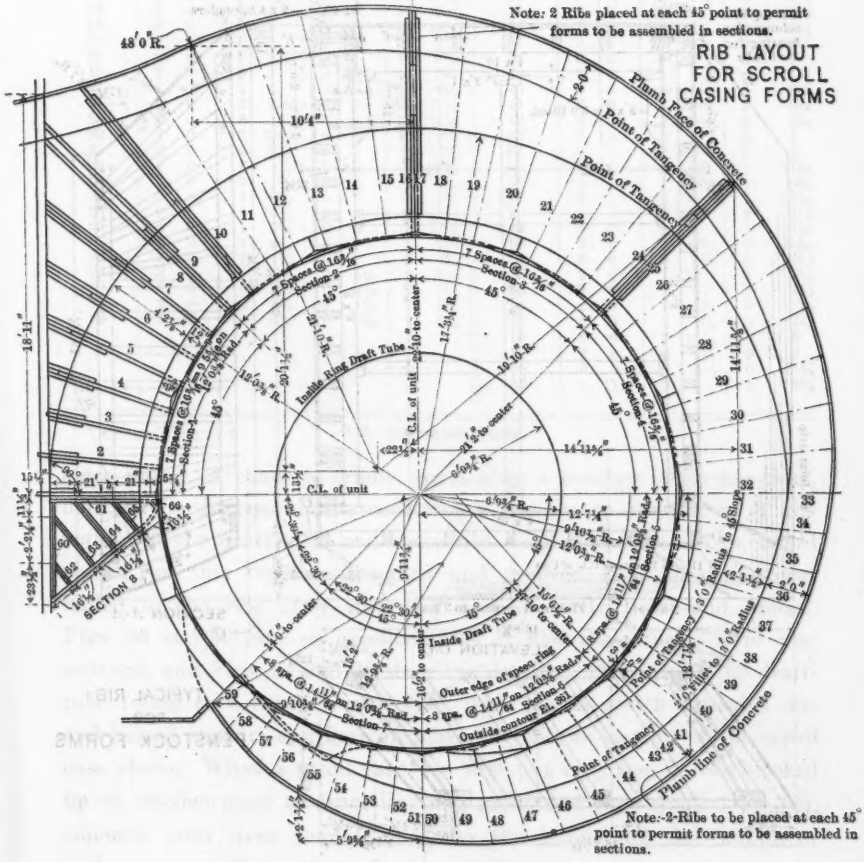


FIG. 35.

In detail, these complicated forms were built as follows: All the ribs of the scroll casings (Figs. 35 and 37), were made according to the drawings furnished, and were then set up in the shop in their proper positions, as laid out, and sheeted with 1-in. material. The

detailed dimensions for the ribs are given in Table 6. On the curved portions, 1 by 1-in. lagging was used, in order that it might bend easily and conform to its intended shape. The whole scroll casing was built in the shop in eight sections in order to facilitate handling

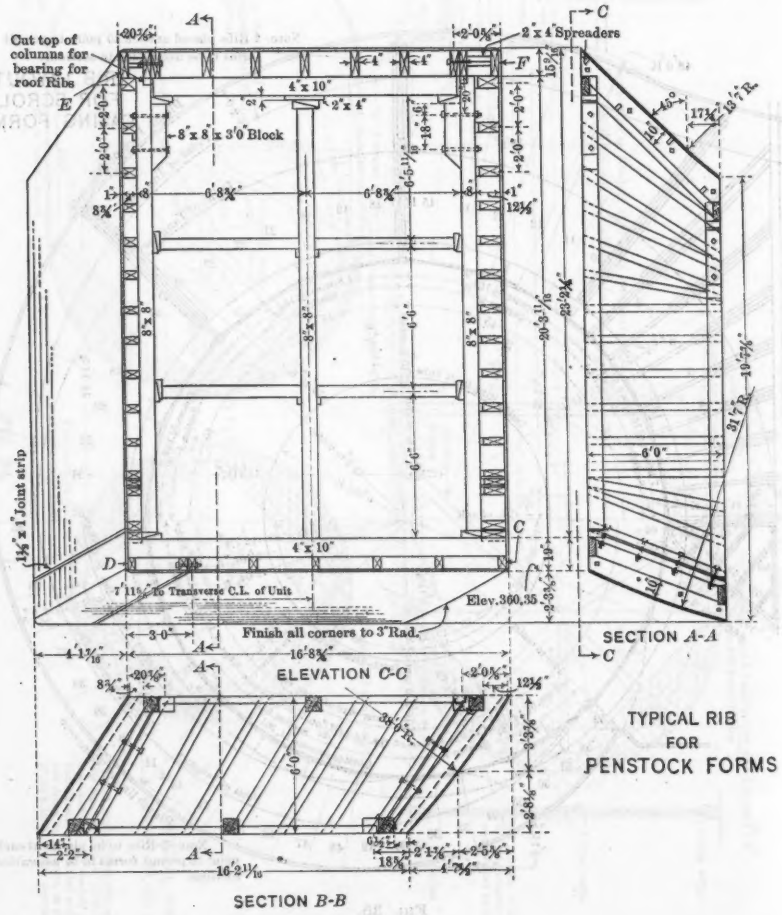


FIG. 36.

and erecting, each section being complete in itself. For the draft-tubes (Figs. 34 and 38), all the ribs were made and fitted together on the form-shop floor; they were then taken down and hauled to the dam and re-assembled on a platform from which a derrick could lift and

TABLE 6.—Detailed Dimensions for Ribs of Scroll Casing Forms.

Rib. No.	A		B		C		D		Rib No.	a		b		c	
	Ft.	In.	Ft.	In.	Ft.	In.	Ft.	In.		Ft.	In.	Ft.	In.	Ft.	In.
10	2	21½	5	4¼	0	6	12	9⅝*	33	2	0	5	3⅝	4	0
11	2	1	4	7½	0	6	11	11⅜*	34	2	0	5	0¾	3	9½
12	2	0	4	4	0	6	11	6⅞	35	2	0	4	10	3	7½
13	2	0	4	2	0	6	11	4⅞	36	2	0	4	7¾	3	5¼
14	2	0	4	0	0	6	11	2⅞	37	2	0	4	5½	3	3¼
15	2	0	3	9½	0	6	11	0⅝	38	2	0	4	3	3	1
16	2	0	3	7	0	6	10	9⅞	39	2	0	4	1	2	11
17	2	0	3	7	0	6	10	9⅞	40	2	0	3	11¼	2	8¾
18	2	0	3	4½	0	6	10	7⅛	41	2	0	3	9⅝	2	6⅜
19	2	0	3	1¼	0	6	10	4⅛	42	2	0	3	9⅝	2	6⅜
20	2	0	2	10¼	0	6	10	1⅛	43	2	0	3	8	2	4
21	2	0	2	7¼	0	6	9	10⅛	44	2	0	3	6	2	2
22	2	0	2	4¼	0	6	9	7⅛	45	2	0	3	3½	1	11¾
23	2	0	2	1¼	0	6	9	4⅛	46	2	0	3	1¼	1	9¾
24	2	0	1	10½	0	6	9	1⅜	47	2	0	2	1½	1	7¼
25	2	0	1	10½	0	6	9	1⅜	48	2	0	2	8	1	4½
26	2	0	1	7¾	0	6	8	10⅝	49	2	0	2	5	1	1¾
27	2	0	1	4¾	0	6	8	7⅝	50	2	0	2	1⅝	0	10½
28	2	0	1	1½	0	6	8	4⅝	51	2	0	2	1⅝	0	10½
29	2	0	0	10¼	0	5½	8	1⅛	52	2	0	1	11	0	7½
30	2	0	0	6¾	0	3⅝	7	9⅝	53	2	0	1	8	0	4½
31	2	0	0	3¾	0	1⅞	7	6⅝	54	2	0	1	4¾	0	1½
32	2	0	0	0¾	0	0⅜	7	3⅝	55	2	0	1	1½	0	1¾
									56	2	0	0	10½	0	5
									57	2	0	0	7¼	0	8
									58	2	0	0	3½	0	11¾
									59	1	11½	0	0	1	3¾

*R. generates ellipse.

place them. A concrete cradle, containing a number of anchor-bolts, had previously been built, conforming exactly to the curve of the bottom of the draft-tube, and these ribs were placed on a 1-in. board resting on this cradle. Tongued and grooved 1-in. sheathing was nailed to the ribs after they had been properly lined and braced. Figs. 33 and 39 give an excellent idea of how these forms were constructed, and Figs. 34 and 38 show the details. The tops of the draft-tube forms were covered with 2 by 6-in. planks which extended out and helped support and keep in place the bottom section of the scroll case above. When a scroll case was first put in place, it was blocked up on wooden posts to exact line and grade and then small 1 by 3-ft. concrete piers were poured up under the joints and the middle of each section. These piers also contained anchor-bolts, and after the concrete of the piers had set, the forms were bolted tightly to them. They were also wired down to hook-bolts set in the concrete below, to keep the form from floating. Penstock forms, Fig. 36, were made in sections, 6 ft. long, in the carpenter shop; they were then knocked down and transported on cars to the dam, where they were re-assembled

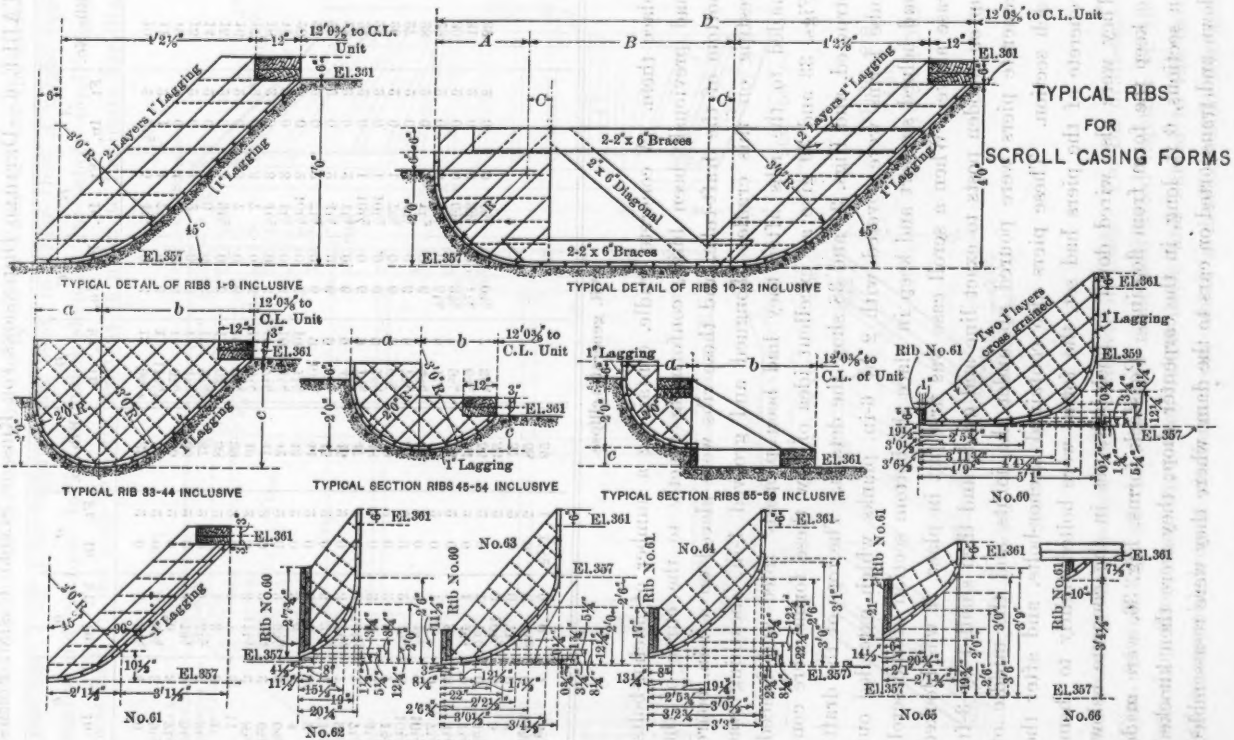
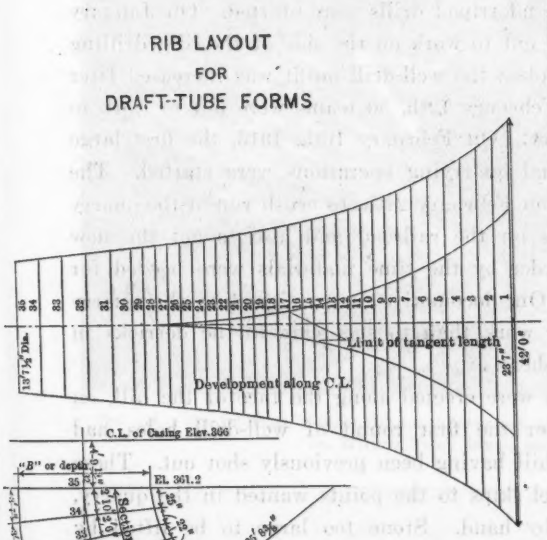

TYPICAL RIBS
FOR
SCROLL CASING FORMS
TYPICAL DETAIL OF RIBS 1-9 INCLUSIVE
TYPICAL DETAIL OF RIBS 10-32 INCLUSIVE
TYPICAL RIB 33-44 INCLUSIVE
TYPICAL SECTION RIBS 45-54 INCLUSIVE
TYPICAL SECTION RIBS 55-59 INCLUSIVE
No.60
No.61
No.62
No.63
No.64
No.65
No.66
2-2"x 6" Braces
2"x 6" Diagonal
1" Lagging
2 Layers 1" Lagging
Two 1" layers cross grained
El.361
El.357
El.359
45°

FIG. 37

on a platform adjoining a derrick which could pick them up and place them. A double cradle with anchor-bolts in it was prepared beforehand, in the same manner as the cradle for the draft-tubes. Types of drawings from which these forms were built are Figs. 34, 36, and 37.

The system of building complicated curved forms on a large laying-



RIB NUMBER	A LENGTH OVER-ALL	B DEPTH	C RADIUS	D TANGENT LENGTH
1	41'7"	18'0"	9'0"	23'7"
2	38'2¼"	18'0"	9'0"	20'2½"
3	35'7"	18'0"	9'0"	17'7"
4	33'6"	18'0"	9'0"	15'6"
5	31'7¼"	18'0"	9'0"	13'7½"
6	30'0"	18'0"	9'0"	12'0"
7	28'5⅛"	18'0"	9'0"	10'5½"
8	27'1⅛"	18'0"	9'0"	9'1½"
9	26'1⅛"	18'0"	9'0"	8'1½"
10	25'2"	18'0"	9'0"	7'2"
11	24'4"	18'0"	9'0"	6'4"
12	23'7"	18'0"	9'0"	5'7"
13	22'10"	18'0"	9'0"	4'10"
14	22'2"	17'11"	8'11½"	4'3"
15	21'7"	17'10"	8'11"	3'9"
16	21'0"	17'9"	8'10½"	3'3"
17	20'5¼"	17'8"	8'10"	2'9¼"
18	20'0"	17'7¼"	8'9⅝"	2'4½"
19	19'6"	17'7"	8'9¼"	1'11"
20	19'1"	17'5½"	8'8¾"	1'7¼"
21	18'8"	17'4"	8'8"	1'4"
22	18'3"	17'2½"	8'7¼"	1'0½"
23	17'11"	17'1"	8'6½"	0'10"
24	17'6⅛"	16'10¼"	8'5¼"	0'8"
25	17'3"	16'9"	8'4½"	0'6"
26	16'11"	16'7"	8'3½"	0'4"
27	16'8"	16'5"	8'2½"	0'3"
28	16'5"	16'3"	8'1½"	0'2"
29			8'0½"	
30			7'11"	
31			7'9⅝"	
32			7'6½"	
33			7'3½"	
34			7'0⅜"	
35			6'9⅜"	

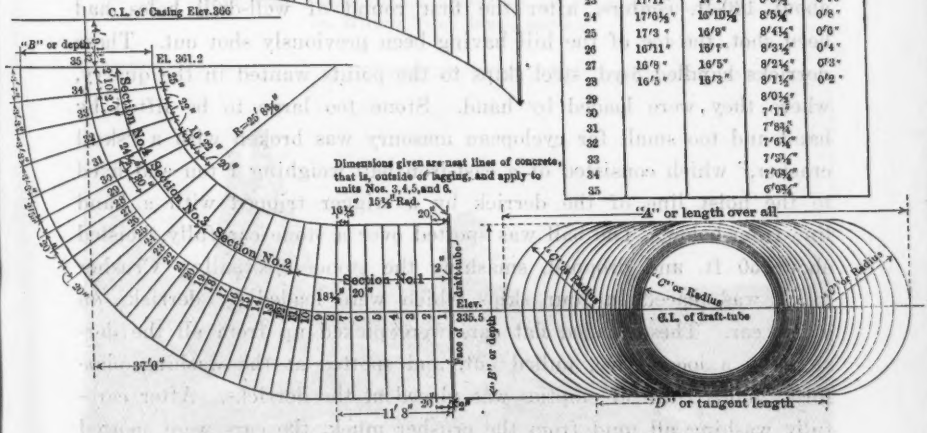


FIG. 38.

out floor, and also all curved ribs for other forms at the carpenter shop, is recommended on similar work. The shop, however, as previously noted, should be much closer to the job than it was on this work. The concrete-cradle method of setting these large forms is recommended as the best and most accurate.

Quarrying Operations, Zion Quarry.—Fig. 24 shows the general layout of the quarry tracks and plant, and the general relation between the various portions of the work. The actual work of clearing and erecting the camp and plant at Zion Quarry was started on November 11th, 1912. On January 20th, 1913, a force was put to work stripping from the toe of the hill, and tripod drills were started. On January 20th, a well drill was also put to work on the side of the hill, drilling the first round of 40-ft. holes; the well-drill outfit was increased later to a total of three. On February 12th, 50 teams were put to work to strip the top of the quarry. On February 16th, 1913, the first large blast was made, and actual quarrying operations were started. The crushers had been started on February 24th, to crush run-of-the-quarry stone for ballast purposes on the railroad and also to get the new plant in good running order by the time materials were needed for concreting at the river. One locomotive was assigned to the quarry service, and did no other work than to spot cars to the derricks in the quarry and to the crushers.

Five steel guy derricks were erected along the face of the hill, on about 100-ft. centers, after the first round of well-drill holes had been shot, the toe of the hill having been previously shot out. These derricks handled 5-yd. steel skips to the points wanted in the quarry, where they were loaded by hand. Stone too large to be lifted by hand and too small for cyclopean masonry was broken with a "skull cracker," which consisted of a cast-iron ball weighing 1 ton and fixed to the hoist line of the derrick by a trigger tripped with a hand line from below. The ball was spotted over a stone carefully, hoisted about 50 ft. and dropped, smashing the stone effectually. Crusher muck was placed in four skips which were loaded by derricks on a flat car. These loaded flat cars were picked up from all the derricks by a locomotive, pulled out, and spotted at the washing platform, and a train of empties was placed at the derricks. After carefully washing all mud from the crusher muck, the cars were spotted to a large timber guy derrick which picked up the skips and dumped them directly into the No. 9 crusher. The dumping device was simply a heavy inclined platform, on which the skip was lowered and prevented from sliding down by a lug under the skip catching a lug on the back of the platform. The skip was held and the contents slid out. After passing through the crushers, the stone was

FIG. 39.—DRAFT-TUBE FORMS UNDER CONSTRUCTION, SHOWING CONCRETE CRADLE ON WHICH THEY WERE SET UP.

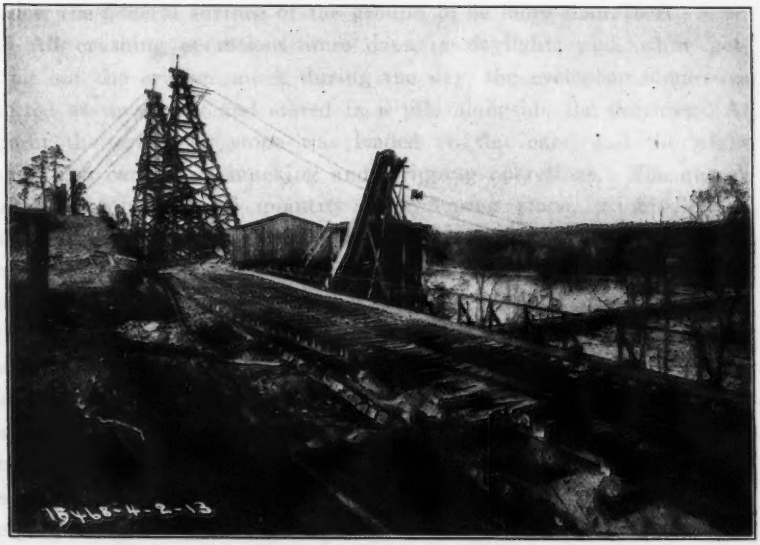


FIG. 40.—END OF HIGH-LEVEL LINE, SHOWING CABLEWAYS, CEMENT WAREHOUSE, AND MIXER BIN.

carried by a belt conveyor to a bin of 1 800-cu. yd. capacity, and drawn from this bin directly into hopper-bottom cars by gravity. (See Fig. 21.)

Well-drills were used for all the heavy work, being augmented in the bottom by tripod drills and Jap hammers. Well-drill holes varied in depth from 40 to 60 ft. These holes were drilled from 15 to 18 ft. back from the face and at about 18 to 20-ft. centers, 4 by 8-in. sticks of Forcite gelatine, 40% dynamite, being used. The largest size of Forcite obtainable was 4 in. in diameter, and the holes originally drilled to 6 in. were reduced to 5¾ in. in order to get the full benefit of the explosive of this size. Only three rounds were taken from the face of the quarry at Zion owing to the heavy over-burden farther in the hill. After they had been cleaned up, about 6 ft. of earth, overlying the rock in a meadow directly in front of the quarry, was stripped back a distance of about 75 ft., and the quarry was then carried vertically downward 60 ft. for a total width of 100 ft. and a length of 450 ft. The seepage from a nearby creek was small and easily taken care of by a No. 6A, 8 by 5 by 13-in., 100-gal. pump. The stone was of excellent quality, and seemed to grow denser with the depth. A well-drill test hole indicated the thickness of the stone below the general surface of the ground to be more than 75 ft.

All crushing operations were done in daylight; and, while getting out the crusher muck during the day, the cyclopean stone was sorted as uncovered and stored in a pile alongside the derricks. At night the cyclopean stone was loaded on flat cars, and the night shift also carried on mucking and stripping operations. The quarry afforded only a small quantity of cyclopean stone, principally on account of the limited space and the necessity for getting out enough crushed stone to keep work going at the dam. It was not possible to take the time to get out the cyclopean stone, because by doing that the crushing would fall behind. In other words, the quarry was such that a sufficient quantity of either crusher muck or cyclopean stone could have been gotten out separately, but not both at the same time. Difficulties and uncertainties in the transportation of coarse aggregates from the outside would not allow that source to be depended on entirely, and reliance had to be put principally on the company's crushers. For these reasons the percentage of cyclopean stone at the dam was much smaller that it would have

been under more favorable circumstances. In this case, the percentage had to be sacrificed to other requirements which were considered more important.

Ellison Quarry.—Ellison Quarry, about 1½ miles east from Zion Quarry, did not prove a success, as stated previously, owing to large clay seams which appeared throughout the rock after shooting. This quarry was worked, however, from March 17th, 1913, when stripping was started, until August 31st, 1913, the same methods of drilling and shooting being used as at Zion Quarry. A locomotive was assigned to its service, as at Zion, and two steel guy derricks were worked. The crusher muck was hauled to the crusher at Zion and unloaded there in the same manner as at Zion Quarry. The stripping at Ellison was done by teams and a steam shovel, there being a heavy overburden against the toe of the hill.

Storage.—Whenever a surplus of crushed stone accumulated in the bins at Zion, it was drawn out into 7-yd. dump cars and stored in a pile alongside a 10-ft. embankment. About 8 000 cu. yd. were stored and afterward picked up by a locomotive crane, operating a ½-yd. clam-shell bucket. By storing materials, the crushing and quarrying operations were carried on continuously, and a shutdown at the dam did not affect the quarries.

Zion Quarry was closed on December 24th, 1913, all the camps were removed, and the plant was shipped out.

Concrete.—As many different classes of material were used in the work, the coarse-aggregate bin, having a capacity of 800 cu. yd., was roughly divided into two sections; all crushed stone was dumped in one end of the bin and all washed gravel and Elmore gravel, consisting of 55% gravel and 45% sand, were dumped in the other end. The sand section of the bin was separated from the coarse-aggregate section by a partition. The crushed stone and gravel were drawn to the conveyor belt at the same time, in about equal proportions, and were elevated into the coarse-aggregate hoppers above the mixers. The sand was drawn by itself and ran into a separate bin, also above the mixers. There was a charging hopper just above the mixers, and all materials were drawn from the bins above into this hopper, where they were properly proportioned.

The mixing of the concrete was watched by an inspector on duty at all times at the mixer plant, and he was constantly in touch with

the chief inspector on the dam, who also with his assistants kept close watch on the concrete coming to the dam.

Three mixtures were used: 1:3:6 in the main body of the dam and the heavy sections of the power-house; 1:2½:5 in the cut-off trench and bottom 2 ft. of the foundation of the dam; and 1:2:4 around the scroll casings, penstocks, and all reinforcing in the power-house. The greater percentage of crushed stone came from Zion Quarry, the remainder being brought from outside quarries and shipped to the dam by rail. In addition to the crushed stone, 31 034 cu. yd. of gravel were used, principally around the complicated curved forms in the power-house and in filling the stream-control openings.

In general, as shown by the analyses of a great many different mixtures used in the work, there was about 10 to 15% too much fine aggregate used. The writers, in view of the results obtained, however, do not consider this a bad feature. There were no honey-combed walls in either the dam or the power-house, and only a few seepage spots appeared in any places except at the vertical expansion joints, the showing in this respect being excellent. When broken off in large chunks, the concrete was dense and had a uniform texture; and when the forms were removed, the outside appearance of all walls was good, no finishing being necessary except in spots where pieces had been broken out by bolts, etc.

Expansion Joints.—At first, expansion joints were placed 108 ft. apart, and after two of them had been started, the distance was cut down to 72 ft. The 108-ft. block proved a little too large to complete satisfactorily as the concrete at one end set before it did at the other. All joints were painted with a heavy coat of hot Barrett pitch. These joints were very effective, as far as could be noted at the end of one winter, and no cracks of any kind developed in the dam.

Handling and Placing the Concrete.—The concrete was dumped from the mixers into 2½-yd. buckets on small cars, or directly into chutes. The cars were hauled by mules to the cableways, two cars being used for each mixer. There were four tracks under the cableways, drawing closely to each other in pairs at the mixers. See Figs. 22 and 40. The buckets were picked up by the cableways and placed at the points needed. No. 1 mixer also dumped directly into a chute. Chutes were used wherever possible to deliver concrete to buckets traveling on small cars operated with a hoisting engine. A large

portion of the power-house concrete was passed through chutes, and the system when properly laid out is most effective. The chutes delivered the concrete to buckets on cars which were taken to the dam, *via* the Low-Level Line and the up-stream coffer, by a dinky locomotive. This method was as effective and rapid as any used on the job. A great deal of the concrete was dumped directly from the cableways, and this method was fairly rapid. However, the maximum 10-hour run with cableways was seldom more than 400 cu. yd. The writers believe that cars and derricks, where it is possible to use them, are more efficient and rapid than cableways. Where narrow forms are to be concreted, the cableway is exceedingly slow and should not be used if any other method can be found.

Cyclopean stones were delivered directly to the derricks on the coffer-dam, and, between buckets of concrete, were set by the derricks. Each stone was carefully washed and cleaned, and, after it was placed, it was carefully shaken with a bar and bedded. In the dam the percentage of cyclopean stone was small, for the reasons mentioned in discussing the quarry operations. The horizontal joints between pours were always left bedded with as many cyclopean stones as possible, in order to get a thorough bond with the next layer.

A total of 6 482 cu. yd. of cyclopean stone was placed in the dam, or 5.1 per cent. In the power-house foundations there were 1 188 cu. yd.

There were two types of chutes on the work. The first was made of No. 12 gauge metal and was 10 in. in diameter. It was of the semicircular type, and was served by a special bottom-dumping car at the mixers. The mixer dumped into the car, and the car was pushed by hand over a hopper to the chute, and dumped into it. This chute was not a success, the main objection being that it was too small. It clogged frequently, and took too many men strung out along it to operate it and keep it clear. The metal was too light and wore out too quickly. This size and type are not recommended for similar work. This chute was abandoned, and a home-made one was built, which gave satisfaction. It consisted of a trough, built in 6 to 8-ft. lengths, with sides about 18-in. high, and well braced; $\frac{1}{8}$-in. sheet iron was rolled in 8-ft. lengths to a half circle, 14 in. in diameter, and fastened into the bottom of the trough, giving a chute with high sides and a rounded bottom. It was of sufficient size, discharged on slopes of 18° rapidly and without clogging, provided the concrete was wet

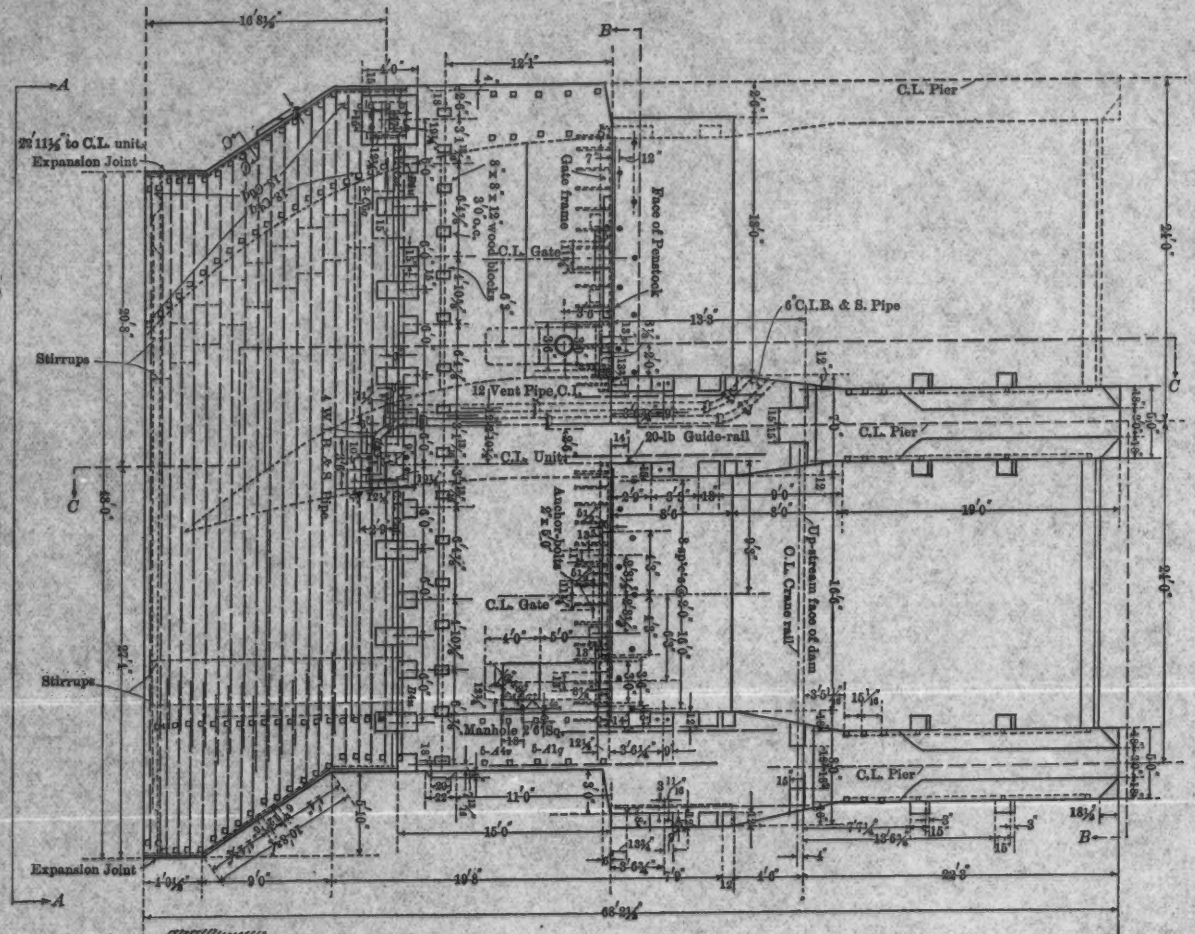
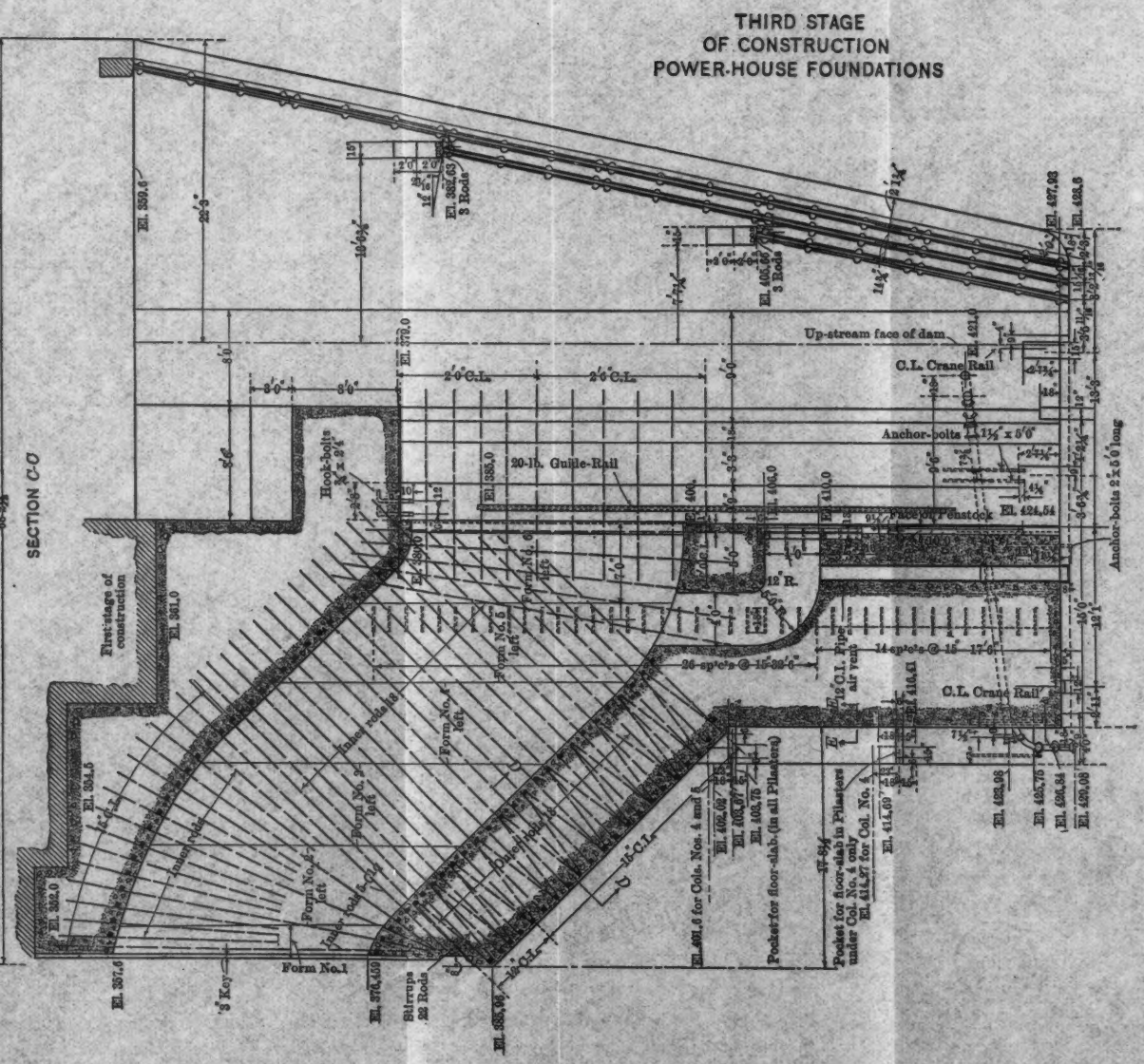

A
B
C
C.L. Pier
Expansion Joint
Stirrups
C.L. Gate
Face of Penstock
6" C.I.B. & S. Pipe
12" Vent Pipe, C.L.
20-lb Guide-rail
C.L. Unit
Anchor-bolts
Up-stream face of dam
C.L. Crane rail
Manhole
Expansion Joint
SECTION C-C
El. 252.0
El. 234.0

PLATE XXVII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



enough to flow, cost about two-thirds of the price of the first chute, and its life was nearly twice as long. It is to be noted that concrete discharged down a 75 to 100-ft. chute was as good as that which fell only a few feet from the mixer to the bucket, and on dumping the two side by side in the forms no difference could be discerned.

After leaving the mixer platform, the concrete was delivered to a derrick, *via* some one of the routes mentioned, and by it placed in the form. An inspector in the form saw that the concrete was brought up uniformly throughout, no racking being allowed. When it was seen that the form could not be completed in one pour, a temporary saw-tooth bulkhead was put in to prevent thin-edged layers. The inspector saw that all concrete was well worked after being dumped; when it lacked sand, it was worked with shovels and tamped until the mortar had worked its way all through it. All faces against forms were carefully spaded. Two bars were used in bedding the cyclopean stone in the soft concrete.

Before making a new pour on the surfaces of old concrete, they were thoroughly cleaned in the following manner: Two gangs of from 12 to 14 men with foremen, when the work was at its height, were kept cleaning forms only. Whenever possible, the surfaces of the old concrete were cleaned about 12 hours after pouring, while they were still green, all scum mortar and "laitance" remaining on top were carefully picked or shoveled off, and the surface was washed until it was absolutely bright and clean. Great stress was put on this particular feature. Concrete was not allowed to be placed in any form until it was cleaned to the satisfaction of the engineer. After the work got under way, and the amount of cleaning insisted on was fully comprehended by the contractor's superintendent, there was little trouble. Also, after a form had been properly washed and cleaned, a 1:2 mortar was dumped in and swept over the surface with a wire broom just before the concrete was dumped. Leaks through horizontal joints have not appeared in any portion of the dam or power-house, and the writers attribute this to the care exercised in cleaning and working the surfaces of old concrete as described.

Power-House Concrete.—In general, the power-house concrete was poured in the same manner as that for the dam, except that the mixtures varied, as noted elsewhere. Fig. 41 is a general view of the power-house operations on July 2d. The heavy blocks indicated in

the first stage of construction, Fig. 43, were of cyclopean masonry, the remainder being mass masonry. The power-house concreting was prosecuted in four separate stages, and complete details of each of these stages are given on Figs. 43 and 44, and Plates XXVII and XXVIII, respectively. These drawings were followed closely in the field, and were of great assistance. The general plan to be followed was carefully discussed at the beginning of the work, and a line of action definitely decided. Fig. 42 shows the second stage of concreting completed. Fig. 45 shows the second stage completed and the forms for the third stage, that is, the concrete around the penstocks, being placed. In Fig. 46 these forms are shown erected. Fig. 47 shows the down-stream elevation of the power-house foundations, the photograph having been taken from the tail-race. Fig. 48 is a close view of the interior of the draft-tube; the man standing in the center of the opening giving a fair idea of its size at the exit.

General.—On April 12th, the first concrete was poured in Section 1 of the dam, immediately next to the wasteway section. Construction of the wasteway section followed immediately, and then the remainder of the dam in Coffer-dam No. 1, and the heavy foundation blocks in the power-house, shown on Fig. 43 as Stage No. 1.

On May 13th, a night shift on concrete was organized at the dam. On May 26th, concreting in the last block of the dam inside Coffer No. 1 was started, and Fig. 31 shows the general condition at this time. On May 31st concreting was started in Section 6 of the power-house. On June 4th, No. 2 mixer was started and the concreting was begun on the bottom cradles for Draft-Tubes Nos. 5 and 6. On June 13th, the filling of the large natural hole in the foundations under Units 1 and 2 was started. By July 12th the concreting and other work had progressed to such a stage that the water in the river was turned through the tunnels left in Section 1 of the dam. On July 25th, Draft-Tube No. 1 was concreted to grade and ready for the scroll casing. On August 26th, concreting was begun in Coffer No. 2, next to the east abutment, which latter lagged behind on account of a bad mud seam which had to be worked out, and on which concreting was not started until September 22d. On September 5th, the first section of rollway in Coffer No. 1 was reached and poured, this marking the completion to Elevation 406 of a portion of the first section of the dam. On September 27th all the rollway section, inside Coffer

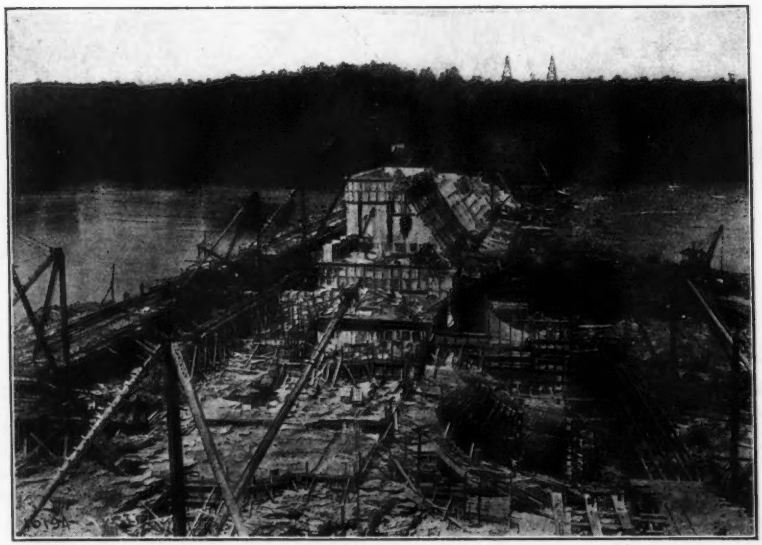


FIG. 41.—CONDITION OF WORK, JULY 2D, 1913. FIRST STAGE OF CONSTRUCTION OF POWER-HOUSE STARTED.

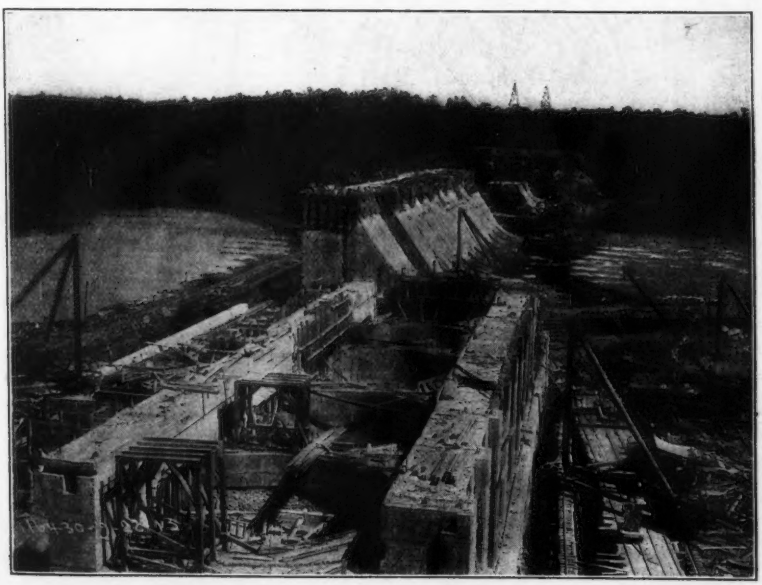


FIG. 42.—GENERAL VIEW OF WORK, SEPT. 22D, 1913. SECOND STAGE OF CONSTRUCTION PRACTICALLY COMPLETED; CONCRETING UNDER WAY.

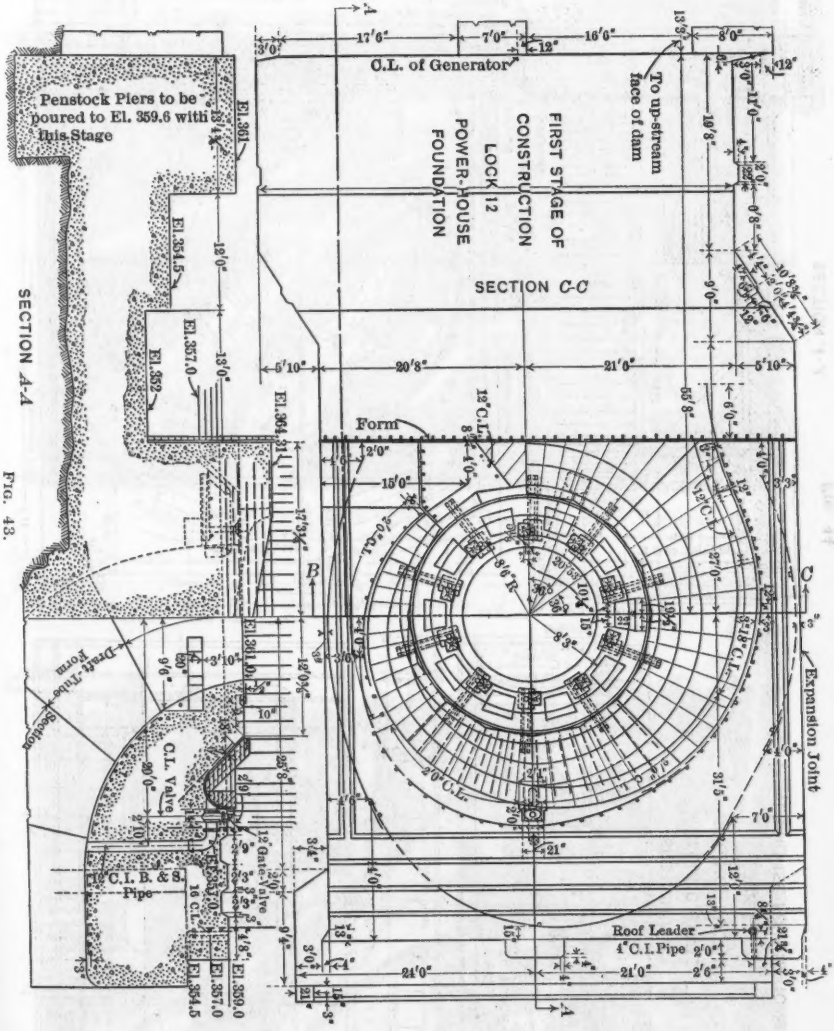

Penstock Piers to be poured to El. 359.6 with this Stage
SECTION A-A
SECTION C-C
FIRST STAGE OF CONSTRUCTION
LOCK 12
POWER-HOUSE FOUNDATION
C.L. of Generator
To up-stream face of dam
Form
Draft-Tube Section
C.I. Valve
C.I. B. & S. Pipe
Gate-Valve
Roof Leader 4" C.I. Pipe
Expansion Joint
El. 361
El. 354.5
El. 357.0
El. 359
El. 364
El. 359.0

FIG. 43.

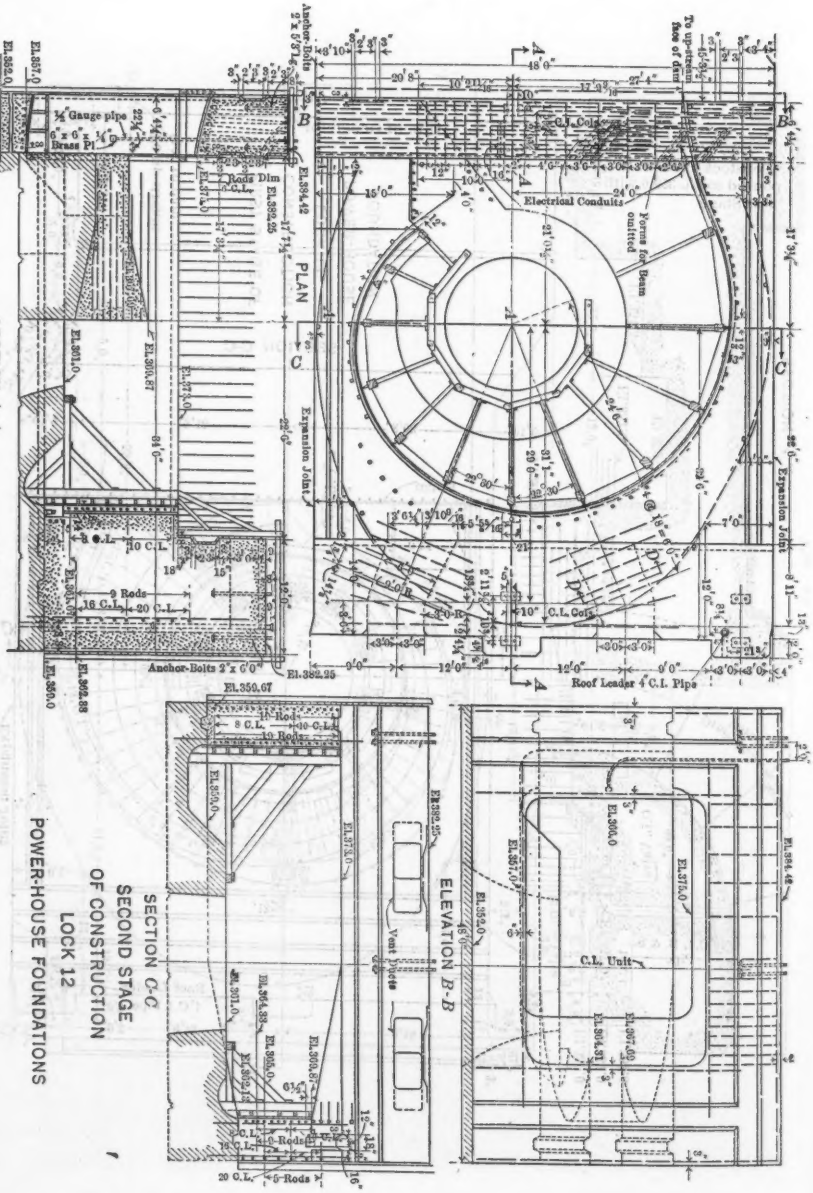

PLAN
SECTION A-A.
SECTION C-C
ELEVATION B-B
Electrical Conduits
Forms for Beam omitted
Expansion Joint
Roof Leader 4" C.I. Pipe
Anchor-Bolts 2" x 6'0"
Gauge pipe
Brass Pl.
Rods Dim
C.L. Unit
Vent Pipes
SECOND STAGE
OF CONSTRUCTION
LOCK 12
POWER-HOUSE FOUNDATIONS

Fig. 44.

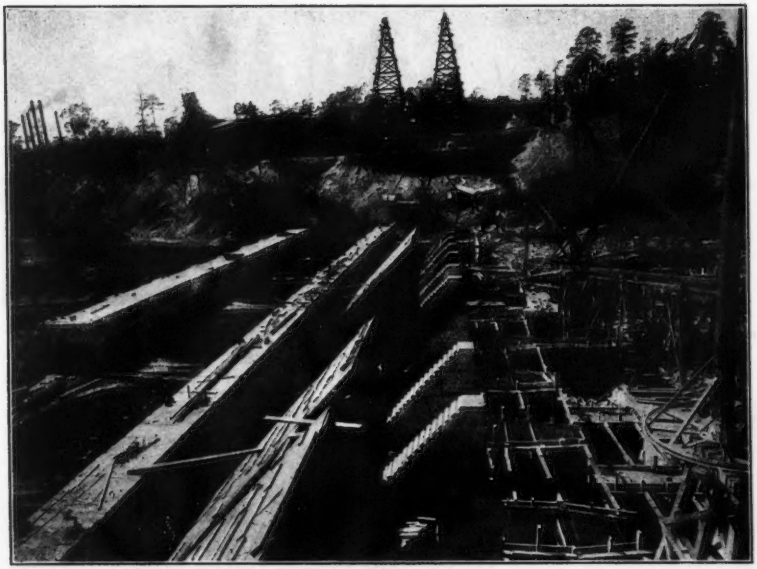


FIG. 45.—CRADLES FOR PENSTOCK FORMS. SECOND STAGE OF CONSTRUCTION COMPLETED.

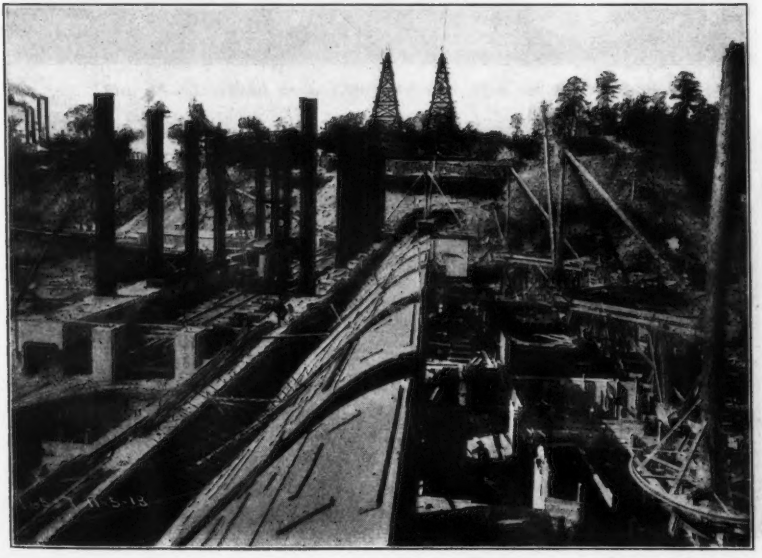


FIG. 46.—PENSTOCK FORMS UNDER CONSTRUCTION. LOCOMOTIVE CRANE SETTING UP COLUMNS OF POWER-HOUSE.

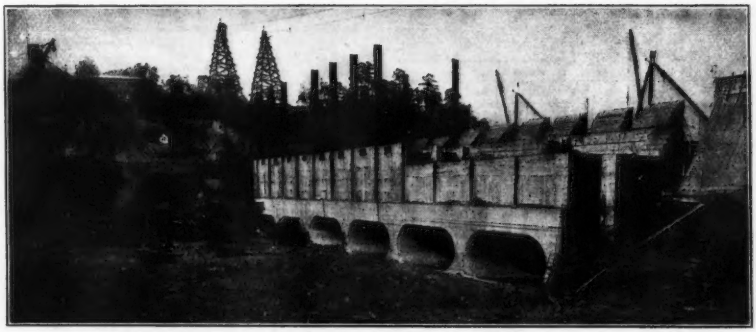


FIG. 47.—DOWN-STREAM VIEW OF POWER-HOUSE FOUNDATIONS, AFTER COMPLETION OF SECOND STAGE OF CONSTRUCTION.

FIG. 48.—LOOKING INTO DOWN-STREAM END OF DRAFT-TUBE.

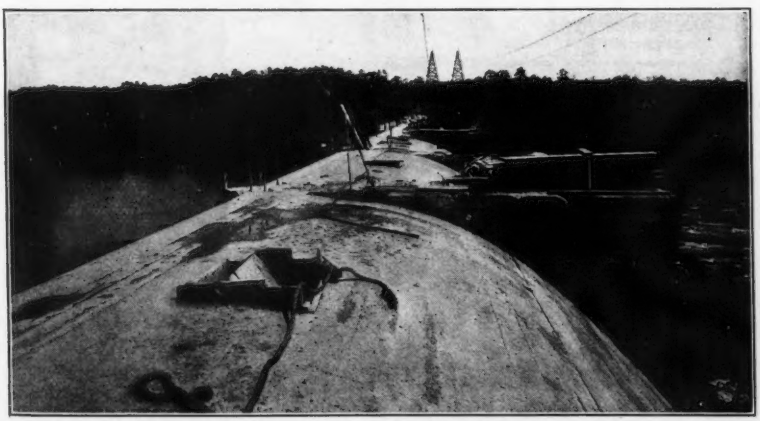
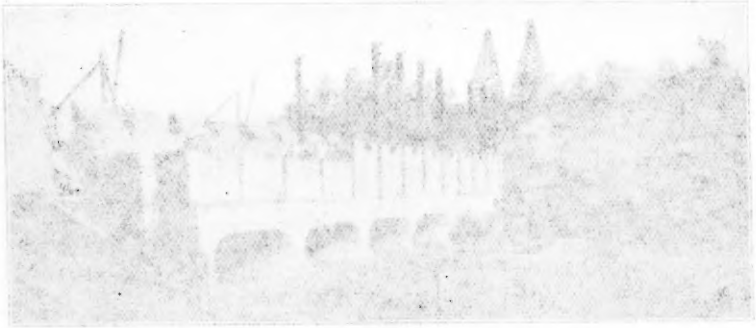


FIG. 49.—CREST OF SPILLWAY BEFORE CONSTRUCTING PIERS.



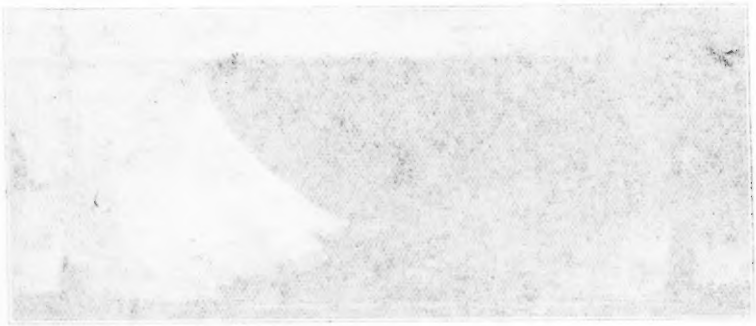
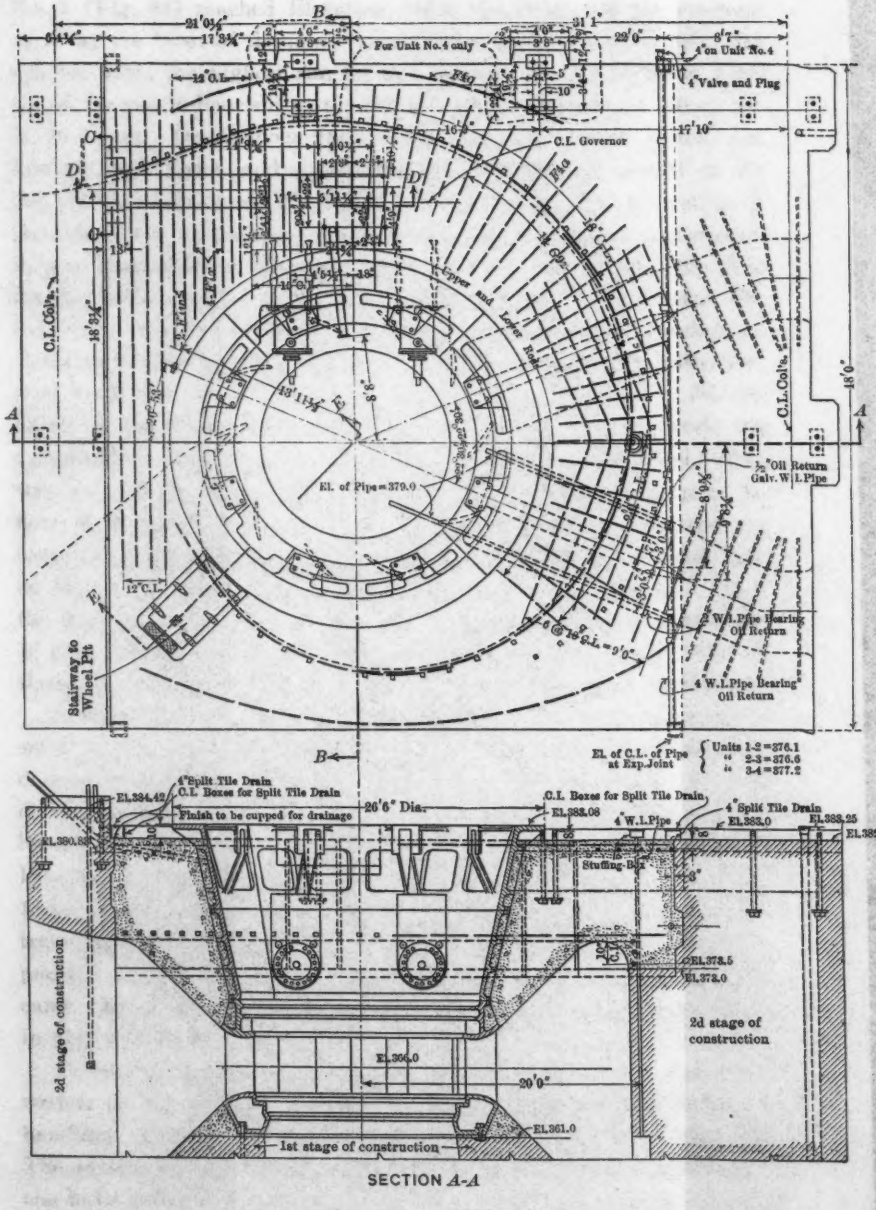
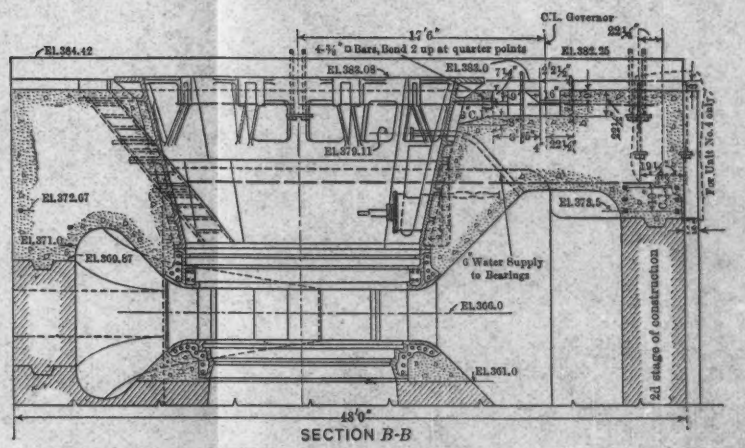


PLATE XXVIII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.



SECTION *A-A*

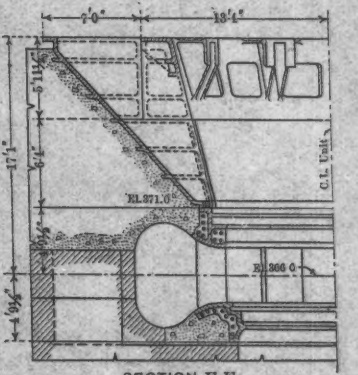


SECTION *B-B*

SECTION *D-D*

Note:-
Bars at Wheel casing to be placed not less than 4" from face of concrete Cast-Iron Boxes for split tile drain to be placed as shown at expansion joint between Units 1 and 2, 2 and 3, 3 and 4.

SECTION *C-C*



SECTION *E-E*

FOURTH STAGE
OF CONSTRUCTION
POWER HOUSE FOUNDATIONS

UNIV. OF MICH.

No. 1 (Fig. 49) reached Elevation 406.0, the crest, and the concrete all along the front of the power-house had reached Elevation 380. On October 13th, the highest run on the concrete was made, being 2 220 cu. yd. for two 10-hour shifts, the day shift alone putting in 1 120 cu. yd. in 10 hours. On October 18th, concreting was started around the first penstock forms, and on October 21st pouring was started on the last section of the dam (No. 9). On November 8th, the concreting of the bottom 4 ft. of the eleven 15-ft. stream-control openings was started. It was completed on November 21st. On November 25th, the dam for the entire length of the spillway was completed to Elevation 406, the crest. On December 20th the power-house concrete reached an elevation of 416 in front, and all the scroll casings, draft-tubes, etc., were ready for the setting of the speed rings. On January 3d, the filling of the stream-control openings was begun, and the work was completed on January 31st. On February 15th, the last wheel casing was set and the last scroll casing poured, completing the operating floor of the power-house to rough grade. On February 28th, the last concrete in the spillway piers was poured, completing the work, and on March 6th, the work of tearing out the mixers was started. From the day that concreting started to the last yard poured, 10 months and 18 days elapsed, and 187 802 cu. yd. of masonry were placed. Fig. 49 shows the finished rollway section before the piers were constructed.

Spillway Piers.—The spillway piers above Elevation 425 were so small in plan and contained so much iron framework that it was very dangerous to dump concrete into them from the cableways. A small derrick was rigged up on a standard flat car—the hoist being operated by air, as usual—for the pouring of these small piers in three 6-ft. lifts each. The cableway delivered concrete to this derrick car. Some 18-in. I-beams, which were bought for the purpose of carrying the traveling gate-operating mechanism between the piers, were used temporarily to span the back of the main piers at Elevation 425, and carry a standard-gauge track for the derrick car. These piers are shown in plan and section on Fig. 19.

Discussion of Methods Used.—In general, as previously stated, the writers do not consider cableways as the most satisfactory method of handling concrete, in view of the experience gained on this job. The system of belt conveyors and elevators for concrete materials is one to be avoided, if possible and if space permits, as a fruitful source

of delay. There was one delay of a week on this work, due to a broken elevator belt, and it was expensive. A gravity plant should be built, if it is possible to lay it out. Handling concrete by buckets on flat cars, with dinky locomotives to the derricks, is preferable to the cableway.

The plant, as laid out and used, was effective and, in 1 month, more than 35 000 cu. yd. of concrete were handled with it, but the writers believe that a gravity plant would have been an improvement and more effective. The point which it is desired to illustrate is that a gravity plant should be carefully sought first, and that cableways should not be made the main method of concrete transportation.

Reinforcement in the Power-House.—Complete and detailed drawings were furnished for the correct cutting, placing, and setting of all reinforcement. All curved rods were laid out on a large platform near the tail-race below the power-house, and there bent with a bender to the curves shown on the drawings. All stirrups, etc., were also made there.

Spacing rods were used, to which the regular reinforcing rods were wired. Large nails were driven into the forms, and the rods were wired to them in order to keep them at the proper distance from the forms. In some cases suspended frames were used to hold the rods in exact position until they were concreted. In all cases particular care was taken to keep the rods the proper distance from the forms and properly spaced. All concrete poured around the reinforcement was mixed in the proportion of 1:2:4.

Completing the Work.

Final Closing.—When all concreting in the dam had been finished, except the stream-control openings, and all piers had been concreted to Elevation 417 on the spillway, and the concreting on the power-house and west abutment to Elevation 420, it was deemed safe to close the gates. Fig. 16 shows these gates hung and ready to close. Previous to the closing, when it was seen that the work was reasonably safe, the west eleven 15-ft. openings in Section 1 of the dam were blocked off by placing wooden gates about 7 ft. high in front of them, one at a time, and a concrete wall, 4 ft. high and 6 ft. thick, was placed next to the bulkhead. This small wall was built first to control the leakage that came through these 7-ft. bulkheads, the water coming

through being collected in an interior box sump in the concrete and led through the wall down stream in pipes which were afterward grouted under pressure. The main part of the opening was then filled to the same height throughout as this front wall, no trouble from leakage being experienced. Each of these eleven openings was treated in the manner described, except that on the five westerly openings, which were built primarily for a stop-log gate, the permanent stop-logs intended for them were dropped into the bottom 7 ft. of the guides and left in. The bulkheads in the remaining six openings were removed after the concrete had set, in order to give a smooth clean sill for the flap-gate to butt against. This work was started on November 8th, 1913, finished on November 24th, 1913, and served two good purposes. It was done primarily as a precautionary measure; the bottom of these tunnels was very close to the natural bed of the river, from 12 to 18-in. on most of them, and with such a small sill for gates of the flap type to strike against, there was a chance for a stone or sunken timber to lodge against it at the time of closing and give a lot of trouble by preventing the gates from seating properly. With this additional 4 ft., the bottom of each tunnel was brought well above the bottom of the river, and also well above the tail-water elevation. There were two gates to each opening, and of the eleven openings, five were of the stop-log type, shown by Fig. 15, and six were of the flap-gate type, Fig. 16. When all these eleven openings had been filled with from 4 to 6 ft. of concrete as described, the ten smaller 8 by 12-ft. gates, which were of the flap type, were hung and securely closed on December 21st, 1913, and caulked around the outside. No water was running through these openings at the time, their bottoms being only 4 ft. below the tops of the large openings. The closing of these gates was a very simple and easy matter, and they were caulked and inspected from the upper side. On December 21st also, five of the larger openings were closed with the ten stop-log gates. The closing of these gates was easily accomplished and did not raise the water very much, as only a small quantity was running through this particular set, the water stage being very low. These gates were then securely wedged and caulked on the outside, up-stream face, and the filling of the tunnels was started. Next, a light template of the larger type of flap-gates still to be closed was made, and each of the openings was carefully checked with this template to be sure that there were absolutely no obstructions to the closing of the gates.

and that nothing of any kind would foul them while they were being lowered. This check was made several times and just previous to the closing.

Owing to the non-delivery of the penstock steel gates by the manufacturers, it was deemed necessary at this time to drop stop-log gates in front of each penstock opening, there being six penstocks with two gates each. As the time was very short and the weather was threatening, it was not possible to secure timbers large enough to withstand a maximum head, consequently, the logs were dropped into the gate grooves and reinforced by four 18-in. I-beams. Later, logs of the proper size were dropped into the grooves regularly provided, the I-beams and logs were removed, and the steel gates put in. The I-beams were those bought for carrying the spillway operating mechanism from pier to pier. All these stop-logs were in and properly bolted and caulked on December 27th. On December 28th, at 7 A. M., the large flap-gates remaining were lowered to a horizontal position, clearing the water by about 2 ft. and rigged up so that by attaching a loose line on one side of the cables holding it above and cutting another loop the gate would fall, but the cable would be held by the loose line and not catch under the gate as it swung shut. On December 27th, rain commenced, and the river, which had been rising gradually all the while, started to rise rapidly, and it was decided to close the remaining gates at once, everything else being in readiness. The gates were closed in the order and at the times shown in the following:

Gate No.	Time closed.
*12	11:20 A. M.
11	1:10 P. M.
10	1:13 "
9	1:18 "
8	1:22 "
7	1:26 "
6	1:28 "
5	1:47 "
4	1:51 "
3	1:54 "
2	1:57 "
1	2:00 "

* Gate No. 12 was first lowered in order to try out the method. The closing of the dam really started with Gate No. 11.

Each gate, after being cut loose, followed three distinct motions: First, a rapid fall of 2 ft. until it struck the water; second, a very slow swinging in with the current until within 3 ft. of closing; and third, a quick, abrupt slam against the concrete face of the wall. The leakage was very small after closing, and was taken care of as described later.

Before the final closure was made, all the gates in the wasteway section were examined carefully, and raised and lowered to test them out in the dry. Plate XXVI shows this wasteway section. Two sets of these gates were left open, and water passed through them during the period of filling the reservoir and for some time afterward, serving to hold the water on the crest of the spillway a little lower while the tunnels were being finally concreted. The water in the reservoir rose with fair rapidity, due to heavy rains, and flowed over the crest of the dam at 9 A. M. of January 1st, 1914. During the period of filling, a careful watch was kept of the river bed below the dam and in the vicinity of the dam itself for any indications of leaks or springs. None whatever was seen, and the dam and rock were absolutely tight.

Concreting the Openings.—In order finally to fill the openings through the dam, the water pouring over the crest had to be diverted and kept out while the work was being done. This was accomplished by building six gates, 3 ft. high and 30 ft. long, reinforced on the back with hog rods, and dropping them into the gate slots provided for the spillway gates. There was only a depth of about 18 in. of water over the crest at this time. All leakage under these gates was stopped by dumping clay and cinder just outside of them, their seats being about 3 ft. from the front face of the dam. It was then necessary to take care of leakage through the openings below and devise a scheme for filling them effectually. The leakage was taken care of in the following manner: A water-tight bulkhead was built about 8 in. back from the inside face of the final closure gate, and a sufficient number of drainage pipes was run from this bulkhead to the down-stream face of the dam to take care of all the water, the leakage being taken care of in most openings by from two to four 3-in. pipes. These pipes were afterward filled with cement grout under a pressure of from 10 to 40 lb. per sq. in. In order to pour the concrete into the openings directly from the cableway, hoppers 6 ft. square were built around the top shaft of each tunnel, but only one shaft was used

for filling purposes, the second shaft, being of no practical value, was not used. The concrete was dumped directly from 2½-cu. yd. buckets into these hoppers and discharged down the shafts into the tunnels, where it was handled by men with shovels to within from 3 to 6 ft. of the roof, when work was stopped and the concrete allowed to set for at least a day. In the first few openings filled, it was thought best to build a concrete wall about 6 ft. thick of a medium dry concrete, to tamp it thoroughly, and to get a tight positive seal. As the wall did not prove to be water-tight, the method was discontinued. The method finally adopted for securing an absolutely tight seal was, as already partly described, to fill the tunnel throughout its whole length to within from 3 to 6 ft. of the roof, the men carefully spading and working this concrete with shovels so as to make a tight job. After this had set for 24 hours, it was carefully washed and cleaned and then a good flowing gravel concrete was dumped into the shaft, vent pipes having been properly placed, which vents also acted as tell-tales as the filling progressed. The back form was bolted up tight to the curve of the dam, and it was found that the dumping of this concrete down the main shaft would force concrete up and out of the back shaft to a height of 35 ft. or more above the top of the tunnel. The writers are of the opinion that, with a pressure of this kind and with a free-flowing concrete, every corner of the tunnel was bound to be tight, especially after the above described demonstration. As a further safeguard, however, a system of grout pipes was put in, and afterward grout was forced into these to fill any possible cracks.

In the ten small openings on the east side of the river, the same general scheme of filling was followed out, that is, a water-tight bulkhead was built 8 in. from the gate, leakage was carried off by pipes to the down-stream side of the dam, and concrete was poured to within 3 ft. of the top and allowed to set hard. Two 2-in. grout pipes were used, one on each side of the tunnel, and as close to the top as practicable, but below each keyway a T had been placed in the pipes and a nipple used so that its end would get within ½ in. of the top of the keyway. Very little grout was placed in these pipes; in fact, concrete in nearly all cases completely filled the tunnels.

One difficulty which should be guarded against in similar operations, was experienced in filling these tunnels: If the sand used was very coarse, as it was in one or two instances on this work, the concrete

would fail to run, and the tunnel would block itself. The remedy adopted by the writers in this emergency was immediately to dump 2-yd. batches of fairly rich mortar until the whole mass started moving again. As a rule, this remedy was satisfactory, and as soon as the mass started moving it came with a rush. In the worst case encountered, twenty-two 2-yd. batches of mortar were used to seal the top effectually. After grouting and setting a proper length of time, the openings were all tight, and showed very small negligible leaks which shortly clogged themselves.

Section D—General Organization of the Contractor and Power Company on the Work.

General Organization of Contractor's Forces.—The main features of the contractor's organization were as follows: A General Superintendent, with headquarters at the dam, was directly responsible to the New York office of MacArthur Brothers Company. He was also responsible for the execution of the contract, and worked directly with the Power Company's Resident Engineer on the job.

His assistants were: Superintendent of the Dam, Superintendent of Railroad, Superintendent of Quarries, Office Manager, and Master Mechanic. All other parts of the organization were handled under these general departments, each department being complete within itself, yet working closely with all the others.

The Superintendent of the Dam had charge of all day and night forces at work at the dam, except such as came under the Office Manager and Master Mechanic. He had a Night Superintendent, and, on both shifts, walking bosses and foremen in all the various departments of the work.

The Master Mechanic at the dam looked after the proper maintenance and operation of all machinery and plant, and all derrick runners, engineers, firemen, machinists, helpers, blacksmiths, etc., were under him. This department worked in very close relation with the Superintendent of the Dam, and derrick runners, cableway runners, pump men, etc., worked to a large extent under both departments. This relation between the Superintendent of the Dam and the Master Mechanic is not in general a wise one; in order to get the best results, the Master Mechanic should be a subordinate of the Superintendent of the Dam; however, in the case in hand, the scheme

worked satisfactorily. The machine shop was about ⅛ mile from the dam, and a large part of the Master Mechanic's time was spent there.

The Superintendent of the Railroad was in complete charge of all railroad operations outside of the Lock 12 and quarry yards. He had a force of trainmen, yardmen, trackmen, dispatchers, etc., under him, and was held responsible for the movement of all material over the railroad. His headquarters were at the junction of the main line and the quarry spur, 7 miles from the dam, known as Camp No. 4.

The Superintendent of the Quarry had charge of all quarrying and crushing operations at Zion and Ellison Quarries, and the camps at these two places. He had day and night forces, with walking bosses, foremen, a master mechanic, timekeeping and clerical force, commissary force, and all the skilled mechanics, engineers, etc., necessary for the work.

The Office Manager handled all commissary matters, purchases of supplies, timekeeping, bookkeeping and all financial matters. He had at Lock 12 a general office force of bookkeepers, timekeepers, stenographers, clerks, etc. All commissary managers at the various camps came under his jurisdiction, as well as timekeepers and material checkers there.

Commissary Department: Organization and Methods.—The Commissary Department deserves special mention, as it was a most important part of the work. Supplies for all four camps and for about 2 000 men were handled through this Department, all kitchens, cooks, as well as stores, coming directly or indirectly under it. The store at Lock 12 was not very well situated, and should have been larger. The writers believe that, had a little more thought been spent on its planning, it would have brought in better returns than it did. No elaborate stock was carried, and no particular attempt was made to realize profits from merchandising. The writers feel confident that it would pay the expenses of all commissary forces to run a store of this kind along proper lines. A store operated just outside of the limits carried a stock of merchandise and fancy groceries, and, even though big prices were charged, a good profit was made by the owner. The writers would recommend that, on a job of this size, the commissary store be a large airy building, centrally located, have regular counters, shelves, and tables for the display of goods; it should have a butcher shop as one department, and also a cold drink stand. It is to be

noted that the small cold drink side line operated in connection with the commissary practically paid the wages of two clerks alone in the summer, and one gave only about half his time to it. One general commissary inspector should be appointed, and he should be experienced in operating a store of this kind. All the various camp commissaries should be under him, and it should be his duty to inspect and watch constantly the character and class of goods demanded and to fill that demand.

It is astonishing how negro laborers and others on construction work will spend all they make for good clothes, candy, fancy groceries, etc., and a good profit can be made from these. It should be the aim to make these commissaries pay all expenses in connection with them and make up the deficit, if any, in the boarding houses, thus relieving the job of that burden as much as possible.

General.—In general, this organization worked out well, and there seemed to be very little friction; at the height of the work all departments seemed to be imbued with a spirit of co-operation and a desire to make high records on the concrete work.

Labor.—The labor problem proved very troublesome. Negro labor was used almost entirely, although other classes were tried. The negro, with all his unreliable features, proved the most satisfactory, but had to be treated properly to get the best results. A regular labor agent was employed to handle exclusively the Power Company's work. His headquarters were at the dam, and he reported to the Contractor's General Superintendent. He was a man who knew the Southern labor market well and was thoroughly familiar with all conditions. The greater part of the negro labor came from Florida, and it was necessary to keep a constant stream of men coming all the time. Some negro labor was brought from Mississippi, Memphis, Tenn., and Birmingham, Ala. All transportation was paid by the Company, and deducted later from the laborer's wages. If he remained and worked 60 days, he made his transportation, and it was returned to him.

Quite a few foreign laborers were brought, *via* Savannah, Ga., by sea, from New York City, being secured by New York agencies, but they were not satisfactory at the dam. They were then tried at the quarry and did so much better that for a long period the quarry labor camp was almost exclusively of foreigners. However, the loss was so great on the transportation of foreigners from New York that

labor of this class was finally abandoned, and negroes were put in at the quarries on both day and night shifts. The skilled mechanics, hoist runners, machinists, etc., all came from similar jobs scattered all over the South, and were of the type who follow big jobs of this kind around the country. Quite a few carpenters were brought from New York City, principally Swedes, and proved far more satisfactory than the local talent. The local white labor, skilled and unskilled, was absolutely unreliable, and of an exceedingly poor class. This source did not afford any help whatever in solving the labor question, and was effective in only one section of the work, namely, in floating and placing the coffer-dam cribs. This class was entirely ignored, there being only a very few regular employees drawn from it.

General Organization of the Power Company.—Supervising the Contractor's organization on the power-house foundations and dam, the Power Company's organization was in general as follows: Representing the Chief Engineer directly was the Resident Engineer. His responsibilities under the type of contract used were really those of a General Superintendent of Construction rather than those of an Engineer alone, as not only was he held responsible for the proper execution of the plans, proper inspection of all materials in the field, and the mixing and placing, but also for the working of the job by the Contractor in a proper manner, the disposition of his forces, his expenditures, the proper running and sanitation of his camp, the handling of commissaries, the placing and lay-out of the plant, etc. Furthermore, the Resident Engineer was clothed with full powers to back up and force attention to all orders and instructions given by him in the conduct of the work.

Under the Resident Engineer were two Assistant Engineers, a Chief Inspector at the quarry, and an Inspector of the railroad; one Assistant Engineer looking after the details of the office and all the night work in conjunction with the Resident Engineer, and all such general details of the job as the Resident Engineer turned over to him for attention. The other Assistant Engineer, with his assistants, not only looked after the proper handling and placing of the concrete, but at all times kept close track of the Contractor's working organization on the dam to see that it was handling the work efficiently. He conferred constantly and worked almost entirely with the Contractor's Superintendent on the dam, his main duties being to look after the construction

on the dam and power-house only, to see that all features were according to the specifications, and that the forces were properly organized and handled.

A day and night inspection was maintained at the quarry, the Chief Inspector handling the day shift and the inspection work on that shift. He was assisted by a timechecker, clerk, and night inspector. He was held responsible for the proper inspection of all crushed and cyclopean stone, and saw that only clean materials were loaded. He watched the loading at the quarry, saw that all stone was washed before going to the crushers or to the dam as cyclopean, and measured all cars of crushed stone before they were pulled out. He also was held responsible for the proper checking of all materials and labor used by the contractor there, and reported any inefficient handling of men or equipment. The Night Inspector performed the same duties at night, reporting to the Chief Inspector. The Power Company's forces were provided with a comfortable house and office entirely separate from the Contractor's forces. They, however, took their meals at the Contractor's mess hall.

The Inspector of the Railroad performed the same general duties on the railroad as other inspectors, and his assistant also checked time and materials, as already outlined. His duties, however, were not as important as those of the other inspectors.

A force of time and material checkers, entirely independent of that of the Contractor, was maintained in each camp, and vouchers and pay-rolls were approved only on the Power Company's check. All expenditures for material and labor had to receive the approval, in writing, of the Resident Engineer, and this approval was essential to the passage of any voucher for payment.

General day and night forces of inspectors, checkers, timekeepers, etc., were maintained by the Power Company at the dam. All the day inspection work was performed under a Chief Inspector, and he, with his inspectors at the dam and at the mixer, checked all forms, looked after the proper mixing, placing, and working of concrete, the proper preparation of the surface of old concrete, saw that all bolts and reinforcing were properly set, etc. The inspectors on the dam and power-house gave orders to the concrete foremen and walking bosses where they concerned the proper working of the concrete and the cleaning of the surface of old concrete before placing new concrete

on it. They had the authority to prevent any of the Contractor's men from commencing to place concrete if the forms were improperly cleaned out. Great stress was laid at all times on the proper cleaning of the concrete. The mixer inspectors watched the proper proportioning of the mix and the quantity of water necessary, receiving their instructions from the inspectors on the dam. At the mixing plant, there was also a cement checker on each shift.

The night shift organization was handled in the same manner, with an Assistant Engineer in general charge for the Power Company, and a Chief Night Inspector under him. All the survey and lay-out work was handled by two small parties of two or three men each. One of these parties was assigned to the dam and another to the power-house. All levels, lines, grades, etc., were given by these parties. Before and after forms were set up by the carpenters, the lines and levels were given by one of these field parties, and the inspectors would allow no concreting in a form until it was "O. K'd." as to grade and line by the Chief of the Party.

This system of handling the inspection and survey work proved most satisfactory, and is recommended for similar work.

Power-House Superstructure and Equipment.

As previously mentioned, the construction of the power-house superstructure and the installation of machinery was done by the Company's own forces. In October, 1913, the organization of the Company's construction forces was begun, most of the men being taken from Gadsden, where, as noted early in the paper, a 10 000-kw., steam turbo-generating station had been constructed.

A camp was constructed near the main camp of the Contractor's forces, and independent mess halls were provided. Storage yards for structural steel, brick, machinery, etc., were constructed part way down the Low-Level Line, and a traveling gantry crane, later erected in permanent position on the up-stream side of the power-house, was placed in these yards for the purpose of unloading and storing heavy machinery parts.

Steelwork.—On November 4th, the foundations were in such shape in the power-house that steel erection was started. A spur track was built from the Low-Level Line in the bottom, entering the west bay of the power-house on a trestle at Elevation 370, approximately, it being

planned to bring all steelwork, machinery, etc., in on flat cars here and remove it with first a locomotive crane and later with the permanent 100-ton crane. A 30-ton locomotive crane was set up inside the power-house for the purpose of erecting all columns and cross-girders. This crane had a 50-ft. boom with a 30-ft. extension. On Figs. 45 and 46 this crane is shown at work erecting columns, and also the track on which all steel and other materials were brought in. As soon as the steel was in condition to receive the 100-ton crane, it was erected, and the locomotive crane was withdrawn for the purpose of loading cars in the yards. A traveling stiff-leg derrick was then erected on the heavy cross-girders above the 100-ton crane, and it handled the remainder of the steel erection, the 100-ton crane taking the steel from the cars below and delivering it to points under the traveling derrick.

Brickwork.—After riveting was sufficiently advanced, the brickwork was started on a previously moulded concrete water-table. All the window sills, cornices, and ornamental work were of concrete blocks, moulded and cast on the work. All concrete block forms were of cast iron, and the results obtained were very good. Gravel and coarse sand were used for making the concrete. They were mixed fairly dry, being well packed and rammed into the moulds.

Erection of Units.—The foundation rings and shell of the units came in sections. These sections were loaded by the 30-ton locomotive crane on flats and delivered to the 100-ton crane in the power-house. These sections were then assembled on the floor adjoining the foundation on which they were to be set, and put in place, properly lined and leveled up, grouted, and then concreted. The type of speed ring and casing used permitted it to be set directly on the foundation previously prepared. It was blocked into permanent position by jacks and holding-down, foundation blocks, and then solidly concreted. The first speed ring was set for No. 4 Unit on January 8th, 1914, and this unit went into commercial service on April 12th, 1914, the others following rapidly thereafter. The first unit was operated through a temporary switch-board and transformer-house set up outside the main building, in order to start selling this power at the earliest possible moment. This allowed the setting up complete of the main busses, switch-boards, and wiring, without current being on them. Later, when these three units were all installed and everything in readiness, the unit that

had been operating temporarily was pulled out of service, the transformers put in their permanent locations, and the few connections between the unit and switch-board easily and quickly made.

Gates.—Penstock gates were set at the time the units were being prepared for service. Some trouble was experienced on all gate frames on account of the castings being too light and warping or springing. This was true of the spillway gates and the guides for the screens in front of the penstock gates. It is almost impossible to block these to exact line, and, in the cases in question, the clearances were entirely too small. Also, the method of setting gate frames in the original pour of concrete around them should be avoided. In view of the experience gained on this work the writers recommend strongly that in similar designs the following points be observed in the design and erection of cast or structural gate frames:

1. See that ample clearances are allowed, giving wide bearing faces to make up the excess width allowed.
2. All faces should be machined true and square, and should be carefully inspected with that end in view at the mill or factory.
3. Grooves and pockets with proper anchorages should be left in the concrete, the frames should be set up after the mass of concrete is poured, and then carefully lined, anchored, and grouted in place.

A gantry crane which traveled on the front side of the power-house, at Elevation 425, handled and set all the large penstock gates. These gates weighed about 12 tons each, and were operated by large cylinders, using oil under pressure. The spillway gates were all assembled and riveted in the assembly yard, some distance from the dam. They were then loaded on cars and delivered to a scow, built for the purpose, just above the dam. They were towed to the front of the openings, and put in place. These gates were operated by a traveling car which, through a flexible coupling, engaged a gear operating the hoisting mechanism of each gate. This traveling car was in duplicate, one electrically operated, the other steam-driven.

Reservoir Clearing.

Of the 4 700 acres of reservoir land, 1 700 acres were more or less heavily wooded. A thorough investigation showed that there would

be no actual ill effects to the health of the community at large, by leaving this timber standing, but the temperament of the hill people residing in the neighboring territory, and their respect for the law, in so far as it may assist them in obtaining compensation for imagined damages, made it appear prudent to clear the reservoir. This decision was strengthened by the fact that a power company in Georgia, which had not cleared its reservoir, was having endless legal trouble from damage suits filed against it. After consultation with the health officials of the State, it was decided that no timber should be left standing, or in a fallen position, between the 420-ft. contour, the normal lake level, and the 410-ft. contour, the limit of the draw-down. Wherever merchantable timber was within a reasonable distance of the railroads, it was sold, and a quantity of pine timber was converted into charcoal; whatever could not be disposed of in either way was burned on the ground, if possible.

Transmission Lines.

The standard line construction adopted by the Company involved the transmission of current at 110 000 volts and distribution at 22 000 volts. For the former, the double-circuit, galvanized-steel towers were adopted. These towers carry two ground wires of $\frac{3}{8}$-in. Siemens Martin, and six No. 00 copper, 7-strand conductors, drawn to 55 000 lb. per sq. in. in tensile strength. The towers were of three types: Type A used in rough country, weighs 4 600 lb., is 68 ft. high, and has a 10-ft. spacing of conductors vertically and 15-ft. horizontally on the top and lowest arms, and 16 ft. 6-in. on the intermediate arm. The base of this tower is 15 ft. square. Type B towers were used as strain towers; they weigh 4 700 lb. and are of the same general design as the other types except that they have bases 17 ft. square; these towers are used at angles between 3° and 35° and at every $2\frac{1}{2}$ miles on the lines. The Type C tower is used in flat country in connection with Type B towers, the latter being placed at each mile point. These towers weigh 3 600 lb., are 65 ft. high, and have bases 16 ft. square. The ground clearance allowed was 25 ft., and the towers were located so that with a $\frac{1}{4}$-in. coating of ice and 8 lb. wind pressure, this clearance would be maintained. The normal spacing for Type A towers is 700 ft., and for Type C, 600 ft.; the maximum allowed spacing for Type A is 1 200 ft., and for Type C, 900 ft.

The insulators used on the 110 000-volt lines were of three makes: the Ohio Brass, Thomas, and Locke. The strings were made up of 6 units on suspension and 7 on strain. The disks are 10 and 12 in. in diameter. The 6-disk strings were tested to 350 000 volts dry and to 255 000 volts wet, before flash over. The test for mechanical strength consisted of a pull of 5 000 lb. applied in the direction of the axis.

The right of way secured for transmission lines was 100 ft. wide. The towers were set on the middle of this strip, and, 10 ft. from the edge, there was carried an all-copper telephone circuit of No. 8 hard-drawn wire, on 22-ft. creosoted poles, 175 ft. apart. On the telephone lines, porcelain insulators, designed for 6 600 volts, were used on a single cross-arm. The line is transposed every fifth pole.

Wagons were especially designed for the transmission line surveys, and their equipment was very compact. Each wagon contained sleeping accommodations, and cooking and table utensils for a party of thirteen men. A negro cook and a negro who acted as driver and general utility man were carried with the party.

The sequence of construction was as follows: When the right of way was secured, a clearing gang, normally consisting of sixty men and two foremen, cleared the line. Immediately following the clearing gang was the telephone gang, consisting of a foreman and twelve men. This gang erected poles and strung the line complete, tying into the public long distance telephone lines at convenient points, so that at all times there was easy communication with the main office. Following a short distance behind the telephone gang was the foundation gang of seventy-five men and three foremen. This gang dug holes, set the stubs of the towers, and back-filled, or set the rock-anchors, as the case might be. The assembling and erecting gangs of forty men total and two foremen for the two gangs, followed as closely as convenient. The stringing gang, of sixty men and three foremen, hung insulators, and strung the ground wires and conductors, finishing the line ready for service.

Fig. 50 shows part of the stringing gang at work on the line from Lock 12 to Anniston, the right of way being occupied by the tower line and a pole line carrying 22 000 volts. Fig. 51 shows the Sylacauga-Alexander City, 22 000-volt line. In this line, the conductors were carried on wishbone-type cross-arms, and the ground wire on

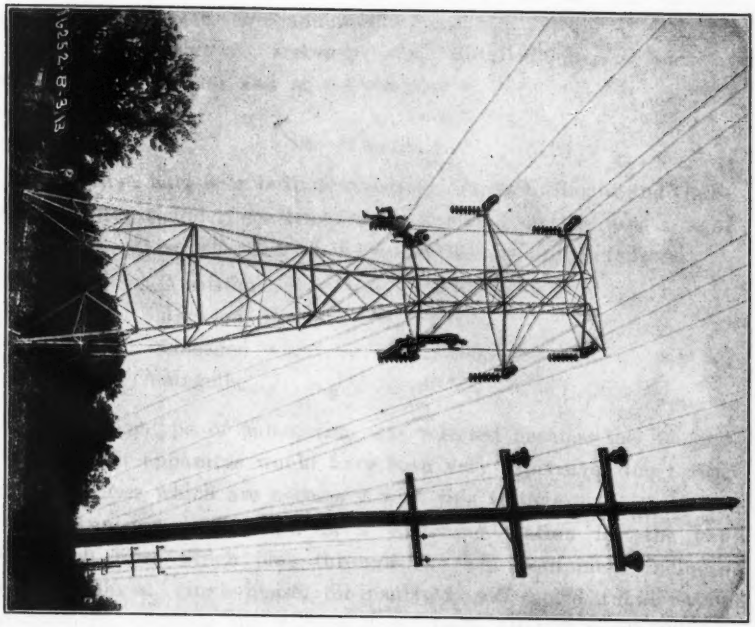


FIG. 50.—STRINGING COPPER CONDUCTORS ON TRANSMISSION LINE OF 110 000 VOLTS.

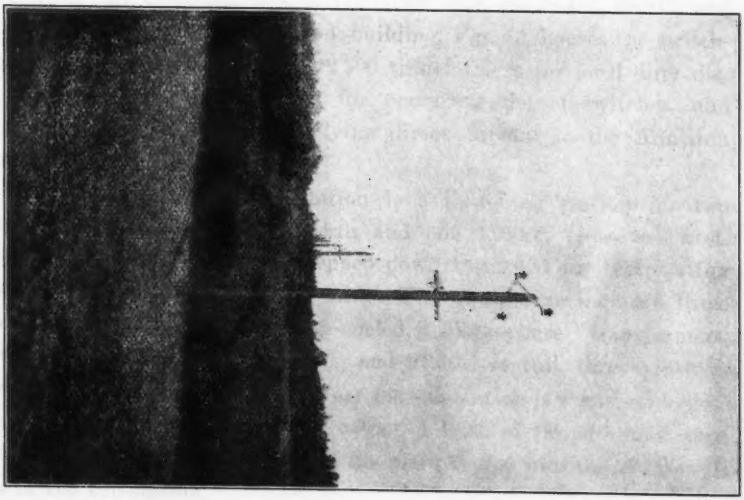


FIG. 51.—DISTRIBUTION LINE FOR 2 000 VOLTS, USING WISHBONE TYPE OF CROSS-ARM.

a bayonet. On all 22 000-volt lines, the telephone line was carried on the same pole with the conductors.

Electrolytic lightning arresters were installed on all lines at the generating stations and at sub-stations.

Sub-Stations.

Sub-stations have been built at Anniston, Jackson Shoals, and Gadsden, and one is under construction at Magella, on the outskirts of Birmingham. The capacities of these sub-stations are as follows:

Anniston	6 000 kv-a.
Jackson Shoals	6 000 "
Gadsden	12 500 "
Magella	40 500 "

The outdoor-type of sub-station was selected because the housing of 110 000-volt apparatus would have been very expensive, due to the large clearances which are necessary with this voltage.

The Anniston Sub-station is a switching station for the two 110 000-volt lines which pass through it. The equipment includes three 2 000-kv-a., single-phase, oil-insulated, self-cooled tubular-type transformers, stepping the voltage down from 110 000 to 22 000 for distribution in the Anniston district. The efficiencies of these transformers at full, three-quarters, and one-half load, are 98.4%, 98.2%, and 97.7%, respectively. A small brick building, Fig. 53, houses the switchboard, three 667-kv-a., 22 000 to 2 300 transformers for local duty distribution, the storage batteries for operating the oil-switches, and a motor generator set for supplying direct current to the Anniston Street Railway.

The Jackson Shoals Sub-station is a switching station for two 110-kv. from the north and south and one 110-kv. from the west. At this point, the voltage is stepped down to 22 000 for distributing lines to the north, south, and west. The equipment includes three 2 000-kv-a., oil-insulated, water-cooled, single-phase transformers, with efficiencies of 98.05%, 97.9%, and 97.5%, at full, three-quarters, and one-half load, respectively. Near the sub-station is the small hydro-electric plant generating at 2 300 volts. A bank of transformers steps this voltage up to 22 000 volts, and the plant is tied into the 22 000-volt bus of the sub-station.

The Gadsden Sub-station is a distributing as well as a step-up station. The generating voltage of the steam plant is stepped up from 2 300 volts to 110 000 or 22 000 volts, or the 110 000 voltage of the transmission lines is stepped down to 22 000, as the case may be. The equipment of the station includes two banks of three transformers each, and a spare-all single-phase, oil-insulated, and water-cooled—each of 2 100-kv-a. capacity. These transformers have three windings, for 2 300, 22 000, and 110 000 volts. The guaranteed efficiencies at full, three-quarters, and one-half load are 98.3%, 98.1%, and 97.6%, respectively.

The Magella Sub-station is a distributing station for three 110 000-volt lines, two from the south and one from the east. The station will step down from the 110 000 volts of the transmission lines to 22 000 volts and to 13 200 volts for distribution, the latter being for distribution into Birmingham. Provision has been made for increasing the capacity of this station from 40 000 to 67 000 kv-a. The equipment includes three banks of three transformers each, and a spare-all single-phase, 4 500-kv-a. capacity, oil-insulated, and water-cooled.

Fig. 52 shows the bus structure of the sub-station at Anniston, which is typical of the others. It is built of towers and girders, this construction having been found to be more economical than an arrangement of towers alone. All the material in the structure is galvanized steel. The weights of the structures are: Anniston, 22 tons; Jackson Shoals, 35 tons; Gadsden, 30 tons; and Magella, 50 tons.

The 110 000-volt busses are No. 00, B. & S. gauge, copper wires, supported horizontally by seven, and vertically by six, 10-in. disk insulators. The busses are 8 ft. apart and are 5 ft. from the nearest steel.

The oil-switches are of the out-door, solenoid-operated, remote-control type, operated from storage batteries in the switch-houses.

Provisions are made at each sub-station for the repair of transformers. There are electrolytic lightning arresters on all 110 000- and 22 000-volt lines, as a protection against the severe electric storms of the region.

It is proposed to standardize future installations of sub-stations into capacities of 3 000, 6 000, and 10 000 kv-a. The bus structure, being made up in unit panels, will be adapted readily to this standardization.

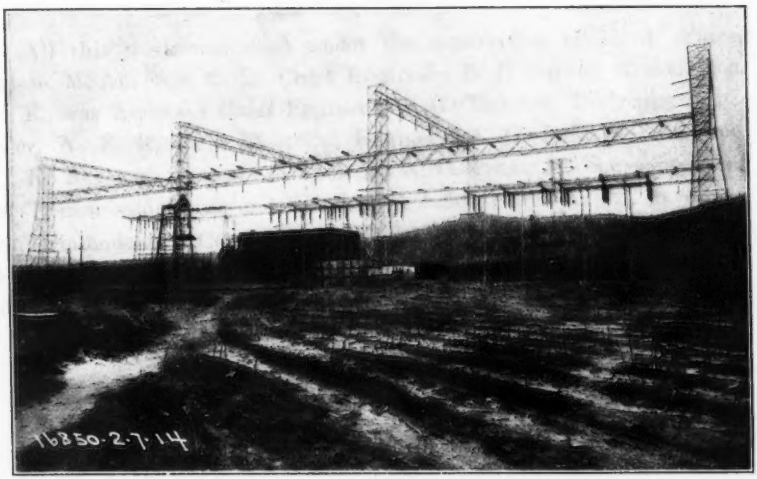


FIG. 52.—OUTDOOR SUB-STATION AT ANNISTON, ALA.; CONSTRUCTION IS TYPICAL OF ALL SUB-STATIONS.

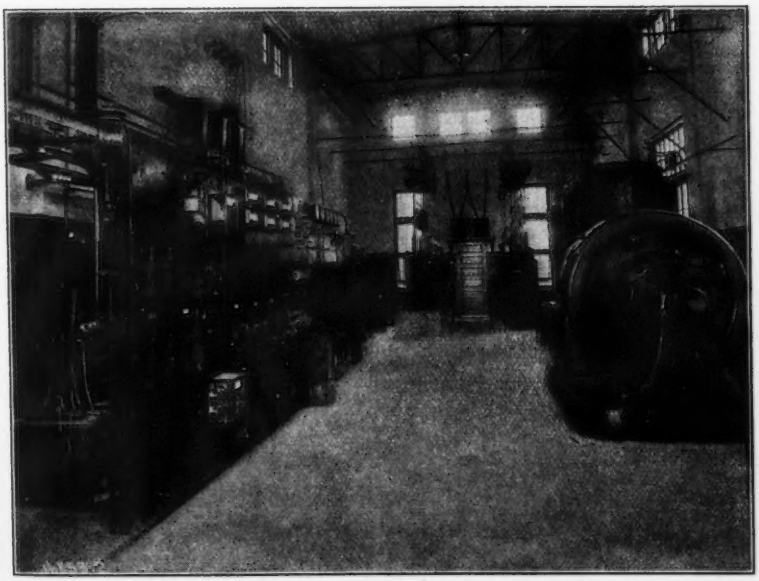
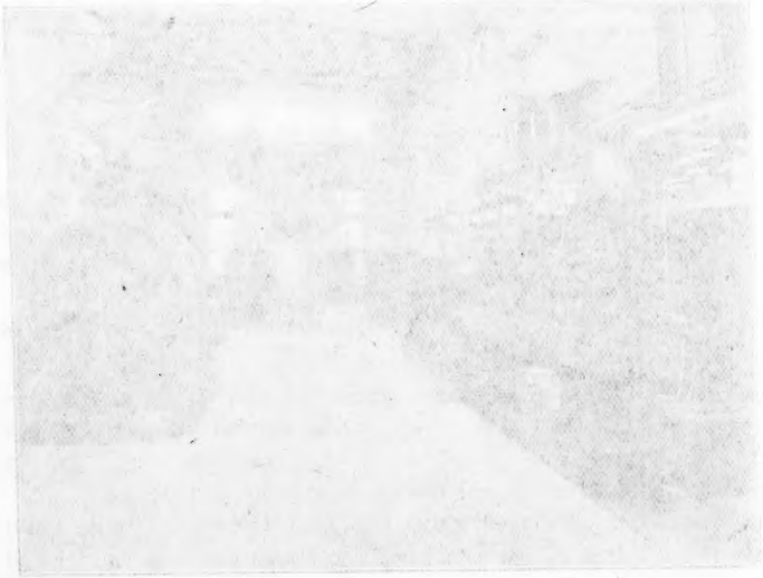


FIG. 53.—INTERIOR OF SWITCH-HOUSE AT ANNISTON SUB-STATION.



Organization.

All this work was done under the supervision of E. A. Yates, Assoc. M. Am. Soc. C. E., Chief Engineer; E. L. Sayers, M. Am. Soc. C. E., was Assistant Chief Engineer, O. G. Thurlow, Designing Engineer, W. E. Mitchell, Electrical Engineer, A. C. Polk, M. Am. Soc. C. E., Resident Engineer at Lock 12, W. C. Cram, Jr., Superintendent of Transmission Line and Sub-station construction, and S. B. Jones, Superintendent of Construction at Lock 12 Power-House and Gadsden Steam Plant. MacArthur Brothers Company were the Contractors for the power-house foundations and dam.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

Discussion.*

By Messrs. T. A. Corry, Frank Foster, H. Deane, and G. F. F. Osborne.

T. A. Corry, M. Am. Soc. C. E. (by letter).—Mr. Lavis' figures show that more than 60% of the Argentine Railways are broad-gauge, and only about 8% are standard. This fact, taken in connection with the lack of ballast and consequent difficulty in keeping the track in good surface, proves unquestionably that if only one gauge is to be adopted, the broad-gauge is the proper one for the Argentine, for Mr. Lavis is quite right in his remarks on the relative stability of broad- and narrow-gauges. Mr. Corry.

For such mountainous countries as Bolivia, Peru, Ecuador, Colombia, and Venezuela, the advantages of the standard gauge over the meter-gauge may be questioned for three reasons:

First. With the meter-gauge the capacity of 95% of the railways required, for the last 30 years and the next 30 years, to handle the available traffic would be quite ample.

Second. Sharper curves can be used on narrow-gauge than on wide-gauge for the simple reason that the rigid wheel base is shorter on both cars and locomotives for the narrower gauge, and the possibility of using sharper curves very greatly reduces the graduation item—a heavy one in traversing the slopes of the Andes, on either the west or east side, in the countries mentioned.

* Continued from April, 1914, *Proceedings*.

Mr. Corry.

Third. It is good business to use a small gun and small shot for small game, especially when expenses are reduced thereby.

It is axiomatic, of course, that a considerable amount of traffic, even in a mountainous country, can be handled at less cost per ton per mile on a standard gauge than on either of the narrower gauges, and the standard is the more economical even if it does cost more to construct.

As to speed, on curves of 100-m. radius (328 ft.), the safe limit is about 28 miles per hour, with the practical elevation slope of 6°, and this speed can be made without difficulty on either 3-ft. or meter-gauge. To refer to a concrete example, take the section of the Peruvian Southern Railway from Mollendo, on the Pacific, to Puno, on Lake Titicaca (Plate XXIX). This road is of standard-gauge, 326 miles in length, and was built in 1868-73, at a cost of 42 000 000 soles (at that time the sol was worth 48 pence, or, say, 97 cents). The maximum gradient was 4%, and the minimum radius of curve 100 m. (328 ft.), and the minimum was used very frequently.

The Mollendo Division is 172 km., or 107 miles, in length, and cost 12 000 000 soles. Besides numerous short tangents, it has six of from 2 to 7 miles in length, aggregating 25 miles. On the remaining 82 miles there are 421 curves, an average of 5 curves per mile. More than half these curves are of only 100-m. radius with an angle per mile of more than 250 degrees.

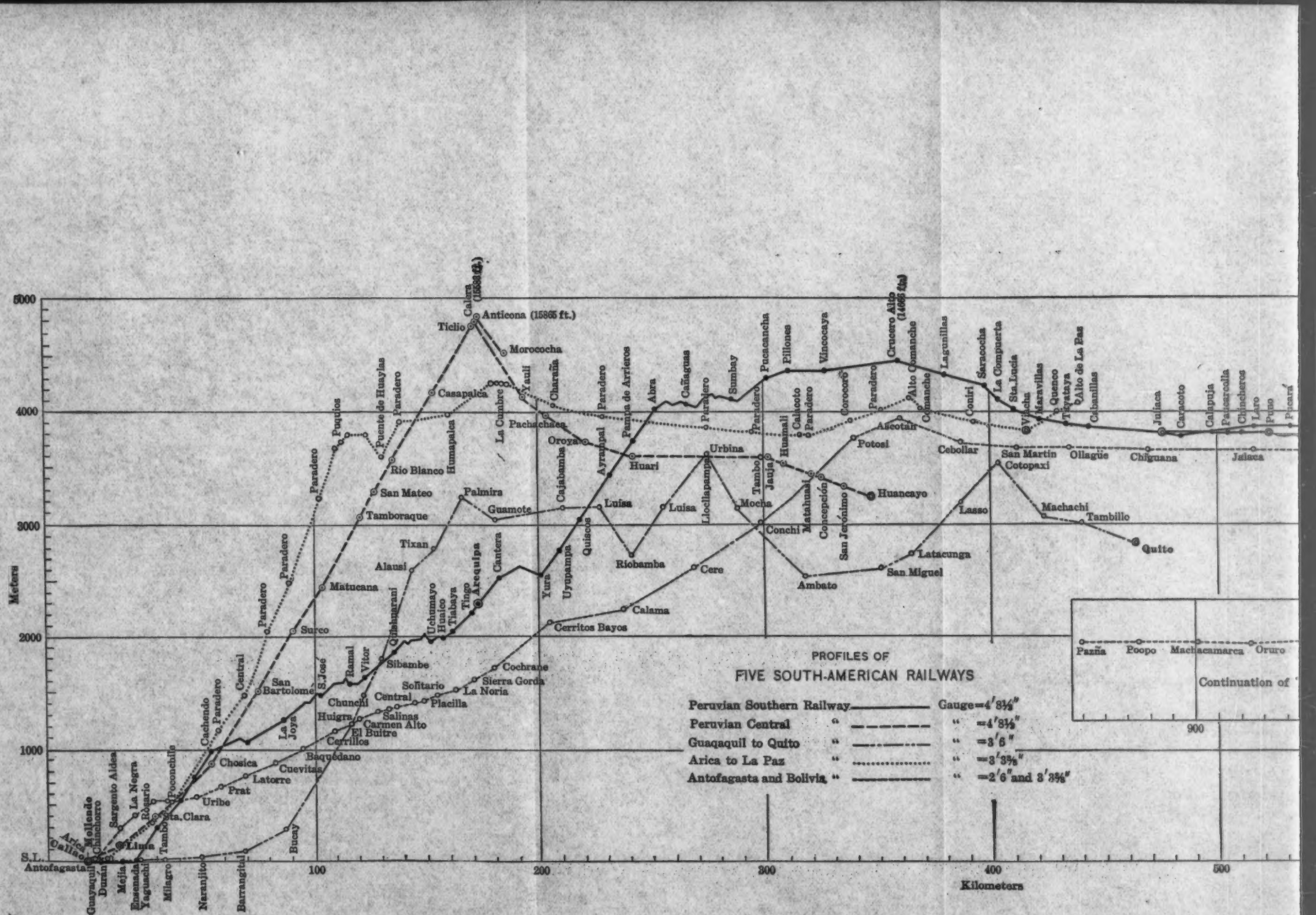
The Puno Division is 352 km. (219 miles) in length, and cost 30 000 000 soles. It has twelve tangents varying in length from 2 to 12 miles, comprising 46 miles, exclusive of those of less than 2 miles in length. The curve section is 173 miles in length and has 783 curves, an average of 4½ curves per mile, the larger number of which are on rough ground, and principally solid rock.

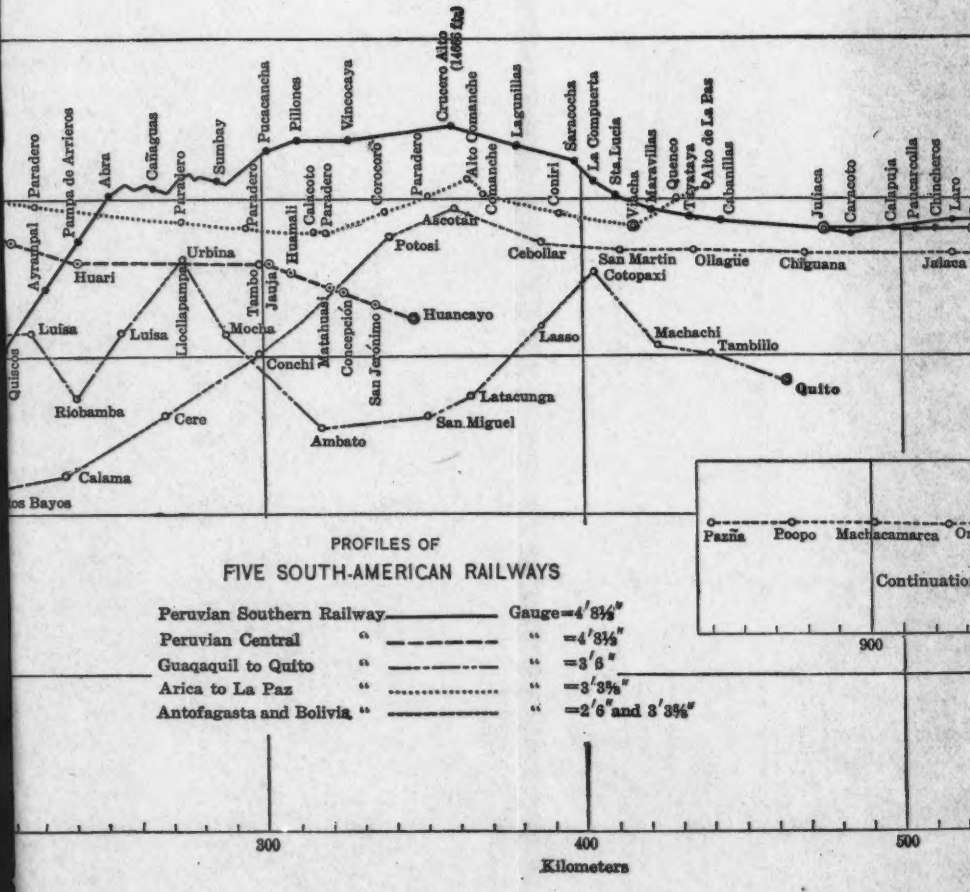
The total daily average tonnage, both ways, has never exceeded 600 in any of the 41 years of the road's existence. The average haul is large, being some 200 miles.

Whether it might have been advisable to have built this road with a 3-ft. gauge, using curves of only 80 m., is debatable on the ground of smaller first cost.

To give another illustration: A few years ago the writer was instructed to make a survey from a point on this road to a river of the Amazon Valley, less than 300 miles in length. The bases given were: maximum gradient, 3%; minimum radius of curve, 120 m.; and gauge, 3 ft. On these bases, the estimated cost reached the sum of $12 500 000, which was a prohibitive one for the traffic and advantages expected.

A resurvey of the roughest part (60 miles), using a gauge of 30 in. instead of 3 ft., with a minimum radius of curves of 50 m.


PROFILES OF
FIVE SOUTH-AMERICAN RAILWAYS
Peruvian Southern Railway Gauge=4′8½″
Peruvian Central " =4′8½″
Guaqaquil to Quito " =3′6″
Arica to La Paz " =3′3⅜″
Antofagasta and Bolivia " =2′6″ and 3′3⅜″
Meters
Kilometers
S.L.
Antofagasta
Arica
Callao
Guayaquil
Mollendo
Durin
Mejia
Ensenada
Yaguachi
Sargento Aldea
La Negra
Rosario
Tambo
Poconchile
Santa Clara
Lima
Milagro
Naranjito
Barrangital
Bucay
Cachendo
Paradero
Chosica
Uribe
Prat
Latorre
Cuevitas
Baquedano
Cerrillos
El Buitre
Carmen Alto
Salinas
Huigra
Chunchi
Central
Placilla
La Noria
Solitario
Sierra Gorda
Cochrane
Sibambe
Ramal
Victor
S.José
La Joya
San Bartolome
Central
Paradero
Surco
Matucana
Paradero
Paradero
Paradero
Chulumani
Uchumayo
Huaico
Tiabaya
Tingo
Arequipa
Cantera
Alausi
Tixan
Tamboraque
San Mateo
Rio Blanco
Humapalca
Puente de Huaylas
Paradero
Paquios
Paradero
Casapalca
La Cumbre
Yauli
Morococha
Anticona (15865 ft.)
Ticlio
Calera (15584 ft.)
Pachachaca
Oroya
Chicla
Paradero
Ayrampal
Pampa de Arrieros
Abra
Cañagras
Paradero
Sumbay
Paradero
Pucacancha
Pillones
Vincocaya
Cruzero Alto (14665 ft.)
Paradero
Alto Comanche
Comanche
Lagunillas
Coniri
Saracocha
La Compuerta
Sta. Lucia
Viacha
Maravillas
Quenco
Tiyatiya
Alto de La Paz
Cabanillas
Juliaca
Carcoto
Calapuja
Pascarcolla
Chincheros
Laro
Pino
Pinaya
Caijebamba
Quiscos
Yura
Uyupampa
Cerritos Bayos
Calama
Cere
Riobamba
Luisa
Guamote
Palmira
Luisa
Huari
Urbina
Lleclliapampa
Tambo
Jauja
Huamali
Caihuco
Paradero
Mocha
Conchi
Matahuasi
Concepción
San Jerónimo
Ambato
Huancayo
Potosi
Ascotan
Cebollar
San Martin
Cotopaxi
Olilague
Chiguana
Jsluca
Lasso
Latacunga
San Miguel
Machachi
Tambillo
Quito
Pazña
Poopo
Machacamarca
Oruro
Continuation of
900


Paradero
Pampa de Arrieros
Abra
Cañaguas
Paradero
Sumbay
Paradero
Pucacancha
Pillones
Vincocaya
Crucero Alto (14665 Ft.)
Paradero
Alto Comanche
Comanche
Lagunillas
Saracocha
La Compuerta
Sta. Lucia
Viacha
Maravillas
Quenco
Tiquitaya
Alto de La Paz
Cabanillas
Juliaca
Carcoto
Calapuja
Pascarcolla
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Latacunga
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os Bayos
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Poopo
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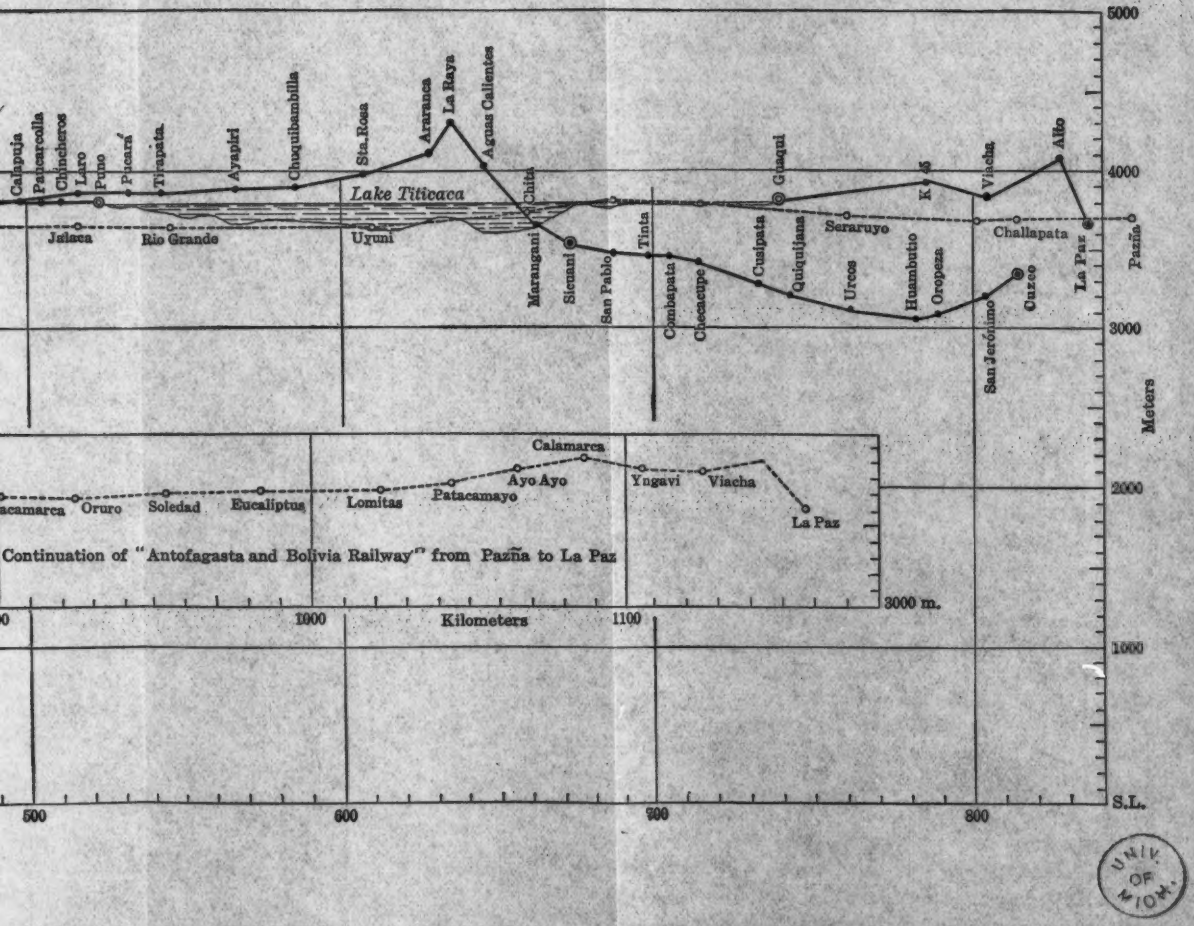
Continuatio
900
PROFILES OF
FIVE SOUTH-AMERICAN RAILWAYS
Peruvian Southern Railway Gauge=4′8½″
Peruvian Central " =4′8½″
Guaqaquil to Quito " =3′6″
Arica to La Paz " =3′3⅜″
Antofagasta and Bolivia " =2′6″ and 3′3⅜″
300
400
500
Kilometers

PLATE XXIX.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
CORRY ON
THE GAUGE OF RAILWAYS.



UNIV. OF ILL.

and a gradient of 5% (for use with Shay locomotives), resulted in a reduction of $3 500 000, or nearly 30 per cent. The principal reduction was due to the sharper curves, by which much heavy work and all the tunnels were avoided. The expected traffic would hardly reach 300 tons per day for the next 20 years. Mr. Corry.

The Antofagasta and Bolivia Railway Company is changing its 30-in. gauge to 1 m., so that now there are in Bolivia somewhat more than 800 miles of railways—all of 1-m. gauge.

In 1912, the total tonnage, including company material, handled was 1 600 710, or nearly 4 400 per day, the average distance hauled being about 130 miles. In view of the great expense being incurred by this road in changing its gauge to 1-m., it is to be presumed that the relative merits of 1-m. and standard-gauge have been carefully considered.

While the writer is not an advocate of narrow-gauge railways, it must be admitted that there are still some mountainous districts, where their use, especially as feeders, is advisable, even if it should prove necessary after a number of years to widen the gauge, and possibly at the same time make other improvements in the matter of alignment, gradients, etc. To put it another way, it may often be inadvisable to build now, for traffic to be handled, say, 40 years hence.

Frank Foster, M. Am. Soc. C. E. (by letter).—In his able paper on railway gauges in South America, Mr. Lavis puts too lightly to one side what appears to the writer to be the principal advantage of the narrower gauges. Mr. Foster.

The Argentine broad-gauge (5 ft. 6-in.) will not allow of shunting with main-line engines on curves of 200 ft. radius, and requires turn-outs with curves of 600 ft. radius for goods trains; and curves well maintained of less than 1 600 ft. radius are rough for main-line running.

In comparison, the meter-gauge stock can circulate on curves of 120 ft. radius, can use turn-outs with curves of 250 ft. radius for goods service, and curves well maintained of 1 000 ft. radius can be taken comfortably at full speed on the main line.

The writer does not wish to imply that broad-gauge stock could not be constructed to circulate under these conditions, for they certainly could, but always at the sacrifice of velocity.

Mr. Lavis quite correctly points out that the broad and standard gauges have enormous advantages in speed over the narrow gauges, but, in fairness, this principal advantage of the narrow-gauge, namely, normal velocity on curves of smaller radius, should be equally extolled. As this is, in the writer's opinion, the unique advantage of the narrow gauges, their use in the Argentine is only justified in mountainous districts, and is to be deplored in the plains, for, without any financial or physical advantages, they have bred costly transhipment

Mr. Foster.

which can only be avoided by a duplication of expenditure, frequently unjustifiable.

Unfortunately, the original railways in the Argentine were constructed of the 5 ft. 6-in. gauge in preference to the standard 4 ft. 8½-in.; and although any idea of correcting this error now or in the future would be financially unsound, as the advantages could in no way compensate the outlay, yet the writer is of the opinion that Mr. Lavis in no way exaggerates the advantages of the standard-gauge as compared with those of the broad-gauge.

An examination of the minimum construction and maximum rolling-stock gauges of the broad-gauge of the Argentine (Fig. 3) shows them to be very similar in dimensions to those of the standard-gauge in the United States, which indicates that in the Argentine no use is made of the wider gauge.

On several occasions the advisability of enlarging the rolling-stock gauge has been under discussion between the officials of the railways and the Argentine Government Railway Board, but it has been resolved, for the reasons given by Mr. Lavis, that there was not sufficient advantage, either in locomotives, coach, or wagon stock, even though the cost of increasing the construction gauge was not prohibitive. For

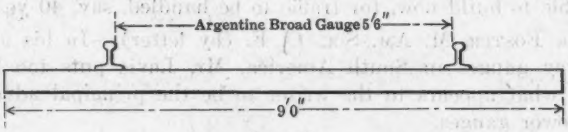


FIG. 13.

instance, the broad-gauge lines of the Argentine at that time had no tunnels, and there are very few over-bridges.

The carrying capacities of the Argentine broad-gauge and the United States standard-gauge are in reality the same, the only appreciable difference between them being the position of the rails, that is, the gauge.

The rail position of the Argentine 5 ft. 6-in. gauge, on 9-ft. sleepers, Fig. 13, represents a very distinct disadvantage to maintenance, as the length of the sleepers outside the rails is not sufficient for adequate support; theoretically, the maximum pressure transmitted by these sleepers is at their extremities instead of under the rails, as is the case with standard-gauge and 9-ft. sleepers, and is at that point 35% in excess of the average pressure instead of 10 per cent. These disadvantages exist, to the writer's practical knowledge, and, in earth ballast, cause excessive pumping at the extremities after rains, and frequent breakage of the sleepers at the center.

In this respect the Argentine meter-gauge tracks, with 6-ft., and in some cases 6 ft. 8-in., sleepers, have better proportions, but the overhang of the rolling stock, shown by Fig. 3, is out of proportion—for

Mr. Foster.

good maintenance on earth ballast—both to the gauge and to the sleeper length. This practically results in greater expenditure in maintenance, rougher riding, and reduced velocity, as compared with the broad gauge. Owing to the general absence of stone in the plains, the majority of the railways in the Argentine are earth-ballasted, except where the traffic is dense or in the few districts where stone exists.

The ratios between the rolling-stock gauge, length of sleepers, and gauge of track are given in Table 30.

The sleepers used in the Argentine are of a specially hard native timber (*Quebracho colorado*), of a density of 80 lb. per cu. ft., which permits the use of a dog-spike connection. These sleepers have a life equal to that of the rails they carry, in many cases exceeding 35 years.

Dense forests of these trees grow in the Northern Provinces of the country, and the soundest timber for sleepers is obtained from the branches sawn in two. The sawn side is laid downward for the bearing on the ballast and the upper side is adzed, as they are rough in section and all the sap-wood rots away entirely within a couple of years. A typical section of one of these sleepers, after the loss of the sapwood, is shown in Fig. 14.

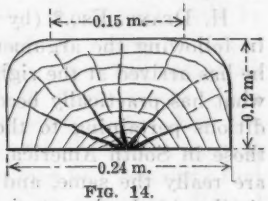


FIG. 14.

Sleepers can be obtained square sawn from the main tree trunk, but are not to be preferred as they frequently split on exposure or spiking. Consequently, up to certain lengths, the cost per cubic foot is more or less constant, but for a length of more than 9 ft. the cost increases considerably.

As there are no native timbers other than the quebracho and similar hardwoods available in the Argentine, apart from the fact that legislation actually calls for the use of the timber of the country, everything is in favor of the use of quebracho in preference to any other timber on these railways.

TABLE 30.—RATIOS BETWEEN THE ROLLING STOCK, GAUGE, LENGTH OF SLEEPERS, AND GAUGE OF TRACK.

	Track gauge.	Sleeper length.	Rolling stock width.	Ratio of sleeper length to track gauge.	Ratio of rolling stock width to track gauge.	Ratio of rolling stock width to sleeper length.
Argentine: Broad-gauge.....	5 ft. 6 in.	9 ft. 0 in.	11 ft. 2 in.	1.63	2.03	1.24
Argentine: Narrow-gauge.....	3 ft. 3⅜ in.	6 ft. 0 in.	10 ft. 6 in.	1.83	3.20	1.75
United States: Standard-gauge..	4 ft. 8½ in.	8 ft. 6 in.	10 ft. 3 in.	1.80	2.18	1.21
United Kingdom: Standard-gauge	4 ft. 8½ in.	9 ft. 0 in.	9 ft. 0 in.	1.91	1.91	1.00

Mr. Foster. Therefore, the Argentine broad-gauge lines will probably continue to use sleepers of this length despite the serious disadvantage to maintenance indicated by the ratios given in Table 30.

On the level plains of the Argentine, curvature has not caused serious inconvenience thus far, but the waste of space in valuable terminals by the length of one-in-eight turn-outs (85 ft.) is an additional serious defect in the broader gauge, whereas, in the writer's opinion, there are no advantages of any kind in the use of the 5 ft. 6-in. gauge in the Argentine, as compared with those of the 4 ft. 8½-in. gauge.

The writer has not entered into the question of methods of transportation, but would point out that on the Buenos Aires Western Railway it is usual to run trains of more than 2 000 tons gross weight with locomotives weighing more than 100 tons. These trains at times attain a length of 2 500 ft. On that railway, wagons with carrying capacities of from 40 to 45 tons represent more than 80% of the total carrying capacity of those in service.

Mr. Deane. H. DEANE, ESQ.* (by letter).—The writer has been much interested in following the arguments set forth by the author, and believes that he has arrived at the right conclusion when recommending for adoption what has practically become the standard and world gauge. The conditions pertaining to the problem in Australia differ somewhat from those in South America, in that the interests of the Australian States are really the same, and there is a tendency toward one or at least a similar kind of control in all matters. In the case of the railways there have taken place, for many years past, annual meetings of the officers and Commissioners who have charge in each State, and the desire always has been to work for one common object, the good of all. In South America there are many independent States, the Governments of which are not actuated by any desire for the common good; at the same time, it must be felt that individual interests would best be served if there were one gauge common to the whole country, for when the railways meet on the borders, interchange would thereby be facilitated.

In spite of what has been said in a few quarters as to the inadequacy of the 4 ft. 8½-in. gauge to meet all conditions of crowded traffic, it must be recognized that a medium gauge suitable for the vastly preponderating ordinary conditions should be adopted—one that is not so wide as to be prohibitive in cost when used in a country where the population is sparse and traffic is light. The 4 ft. 8½-in. gauge has been proved to meet all requirements fairly, and it is doubtful whether much more could be done with any much wider gauge than is achieved in the United States on the 4 ft. 8½-in.,

* Recently Engineer-in-Chief for Commonwealth Railways, Melbourne, Australia.

seeing that to get the full theoretical advantage of very much wider gauges, rolling stock would have to be built of very unwieldy dimensions, and this very unwieldiness would tend to reduce its economic value. Mr. Deane.

The case of India seems to show that a gauge much wider than 4 ft. 8½-in. would sooner or later prove a mistake. Railway construction was commenced in that country on a 5 ft. 6-in. gauge, but after a time that gauge seems to have been found to be inconveniently wide, and recourse was had to the meter-gauge for large extensions. The mileage of the railways on this gauge seems to be about 40% of the whole system. Had the 4 ft. 8½-in. gauge been adopted at the outset, there would probably have been no meter lines. Mr. Lavis has shown that, where the meter-gauge was thought necessary, the 4 ft. 8½-in. gauge meets the conditions quite as well. The writer proved that himself when constructing the Wolgan Valley Railway in New South Wales, and this has been mentioned by the author.

In selecting a gauge as a standard in any country, it should of course be borne in mind, that sooner or later connections will be made with outside countries, when all the evils of breaks of gauge will be experienced if the matter has not been previously adjusted.

There is one argument in favor of the 4 ft. 8½-in. gauge which should not be lost sight of. The present tendency is to adopt the same standards for mechanical details throughout the world, so that when an article is ordered from a distance there may be a certainty that it will fit into its place. The chief manufacturing countries—United States, Canada, Great Britain, and those of the continent of Europe—are engaged in building locomotives and other rolling stock to the 4 ft. 8½-in. gauge, and it is clear that if, in a certain country, there is a sudden demand for rolling stock which cannot be met out of its own resources, orders placed in any of the countries mentioned could be much more promptly met if the country ordering uses the same gauge; whereas, with a different gauge, there would necessarily be special designs, special patterns would have to be made, and so on.

G. F. F. Osborne, Assoc. M. Am. Soc. C. E. (by letter).—The writer has been connected with the administration of a railway comprising a section of 5 ft. 6-in. gauge adjacent to a large port, and a meter-gauge section, farther away, built as an extension of and as a feeder to the 5 ft. 6-in. section. Although the accounts and statistics of the two gauges were kept separately, both were worked as one system by one establishment of administrative and executive officers, and members of the staff in all grades were transferred, as occasion arose, without reference to the section on which they were serving or to serve. The results, therefore, afford a very fair basis for judging the relative operating merits of broad- and narrow-gauge for the conditions there obtaining. Mr. Osborne.

Mr. Osborne.

Both lines are built over flat country intersected with waterways. There was very little cutting (practically none) on either system. The area served by the broad-gauge is densely populated; that served by the narrow-gauge more sparsely so. In both cases the individual inhabitant has a very small traffic value, either by his personal journeys or as a producer or consumer.

Before these lines were built, the country was served by a well-organized system of water carriage, which still exists; and competition is severe. The system has arrived at its present state partly by direct construction, but partly by the absorption, by purchase, of other lines; and their purchase price is now shown in the capital accounts as a lump sum. The broad-gauge section bears the brunt of the expenses due to terminals, has a heavy suburban traffic, and is generally constructed on a higher standard than the meter-gauge. On the other hand, the possible extension of the broad-gauge to replace the meter-gauge (now in progress over some 60 or 70 miles) has led to the construction of bridges having spans of 60 ft. and greater on the broad-gauge standard, and the expense (a considerable item) is charged to the meter-gauge. Another broad-gauge railway reaches the port by running powers over the quadruple portion of the broad-gauge section and two of the four tracks are necessitated by this traffic (which has not been here considered). The meter-gauge stops about 125 miles from the port, and all through traffic is transhipped at the point where the gauge changes.

The system is a network rather than a line, and has many branches. The longest distance run by a through train is 150 miles on the broad-gauge and 335 miles on the meter-gauge. Over the whole system the average haul of a passenger is 23 miles; of freight, 192 miles. Practically the whole staff is housed in buildings belonging to the administration and charged to the capital of the undertaking. Station buildings, houses, platforms, and similar structures are of masonry. The system is soundly built, well equipped, and adequately maintained.

Practically all the passenger stock is fitted with automatic brakes, but this is true of only a small percentage of the freight stock. The busier parts of both sections are worked on the block system, with token instruments; the remainder is worked on the written "line clear, authority to proceed" system. Signaling is normal danger.

The statistics in the tables are for 1912, and are derived from the half-yearly analysis of working the system; the sum or mean of the figures for the two half-years being taken. The error (if any) thus caused is not sufficient to affect the comparative value of the figures.

Before considering the operation statistics, it may be stated that the meter-gauge is being extended through country where the line is not expected to pay in itself for some time after opening, and profits have to be looked for in the increase in traffic brought thereby to exist-

ing portions of the system. The character of the traffic carried is shown by Table 32. Mr. Osborne.

TABLE 31.

Physical Characteristics.	5 ft. 6-in. section.	3 ft. 3½-in. section.
Mileage	512	1 117
Of which is Single-track	397	1 117
Double-track	94	
Quadruple-track	21	
Ruling gradient	1:300	1:200
Sharpest curve: Radius, in feet	1 000	500
Actual capital, cost per mile of line	$90 000	$35 500
Of which approximately is due to:		
Preliminary expenses and general charges	2 250	2 350
Rolling stock	21 750	6 725
Track, works and structures	66 000	26 425
Maximum weight of rail	90 lb.	50 lb.
Width of formation	20 ft.	16 ft.
Maximum permitted weight per axle of locomotives, in long tons	22½	10
Minimum seating space per passenger:		
Width	19½ in.	19½ in.
Floor space	3½ sq. ft.	3½ sq. ft.
Capacity	25 cu. ft.	25 cu. ft.
Standard ballast (stone) per foot of track	6 cu. ft	3 cu. ft.
Cost per tie	$1.46	$0.56
Number of locomotives	253	238
Number of passenger vehicles	750	844
Number of freight vehicles	5 498	4 722
Number of stations on the line (including block cabins outside station limits)	152	209
Number of stations interlocked (including block cabins outside station limits)	69	24

TABLE 32.—PASSENGERS AND PRINCIPAL COMMODITIES CARRIED PER YEAR. (STATISTICS ARE NOT KEPT SEPARATELY FOR EACH SECTION.)

Tons are long tons of 2 240 lb.

Number of passengers	32 975 300
Fibers, raw (mostly loose or not fully pressed), in long tons	1 136 000
Coal, stone, etc	1 107 000
Grain (including flour) and seeds	570 000
Miscellaneous and general merchandise	580 000
Oils, drugs, chemicals, and spices	338 000
Metals and their ores	128 000
Fodder	114 000
Timber, wrought and unwrought	113 000
Salt	106 000
Sugar	84 000
Fibers, spun or woven	80 000
Railway material	72 000

The writer considers that the figures in Tables 32, 33, and 34 prove conclusively that the statement that a narrow-gauge line is more expensive to work than a broad-gauge is not of general application, but needs qualification, depending on the circumstances obtaining in each case. The writer admits the inferiority of a narrow-gauge to broader gauges in one particular only: "Lack of speed." If the speed on the narrow-gauge be sufficiently reduced below that on the broader

Mr. Osborne.

TABLE 33.—Operating Statistics.

Passenger Services.	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Passenger train-miles per year per mile of line	5 139	2 284
Earnings per passenger train per mile (includes mail and express)	$0.99	$0.85
Cost of working a passenger train 1 mile (includes mail and express)	0.58	0.42
Receipts per passenger mile	0.0045	0.0043
Expenses per passenger mile	0.0026	0.0021
Average number of passengers in a train	184	168
Average weight of a passenger train, in long tons	264½	174½
Coal per passenger engine per mile, in pounds	58	39½
Average speed of passenger trains, including stops, in miles per hour	21¼	17⅛
Maximum speed of any passenger train, including stops, in miles per hour	35	25
Freight Services.		
Freight train-miles per year per mile of line	3 482	1 583
Earnings per freight train per mile	$2.10	$1.35
Cost of working a freight train 1 mile	1.22	0.82½
Receipts per freight ton-mile, in long tons	0.0101	0.0107
Cost of carrying 1 ton of freight 1 mile, in long tons	0.0064	0.0065
Average load of freight in a train, in long tons	183	121½
Average gross weight of a freight train, in long tons	551½	352¼
Coal per freight engine per mile, in pounds	87½	62¼
Average through speed of a freight train, in miles per hour	14	12⅜
Ratio of paying freight to total capacity hauled	34.44%	34.35%

Note.—The writer submits these figures for the purposes of comparison only, and not as the best results obtainable on either a broad-gauge or a meter-gauge.

TABLE 34.—General Statistics.

	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Cost of maintenance of track, works, and stations, per year per mile of track	$1 072	$462
Cost of maintenance of track, works, and stations, per year per train-mile	0.211	0.140
Fuel and all locomotive expenses (including wages and renewals), per train-mile	0.198	0.174
Car expenses, renewals, cleaning, oiling, etc., per train-mile	0.076	0.58
All other operating expenses, including station staff, per train-mile	0.194	0.116
Average cost per long ton of coal at engine shed	1.85	2.87
General charges: Audit, stores, medical, police, etc., per train-mile (not separated)	$0.055	.....
Taxes, legal expenses, claims, payments to other lines, etc., ratio to gross earnings (not separated)	5.85%	.....
Average miles run per engine per day	73¼	67¾
Ratio of working expenses to gross earnings	60%	56%
Ratio of net earnings to total investment	5⅛%	5⅛%

Note.—The writer submits these figures for purposes of comparison only, and not as the best results obtainable on either a broad-gauge or a meter-gauge.

Mr. Osborne.

gauge, all ill effects of curvature, stability, track stresses, maintenance, etc., are reduced to the level of the broader gauge, but it follows, partly as a result of this lack of speed, that increasing traffic will overtop the capacity of a narrow-gauge, single line and demand further capital expenditure on doubling, conversion, or other means to increase its capacity, sooner than it will in the case of a single line of the broader gauge.

In the particular case with reference to which the writer has given statistics, he considers that extensions on the meter-gauge were a mistake, not on account of any inherent defect of the meter-gauge itself, but because of the difficulties arising from the break of gauge. The cost of extending on the broad-gauge would probably not have involved an additional expenditure of more than $2 500 per mile, against which would have been the saving in rolling stock detained at transhipment junctions and the cost of transhipment terminals, probably sufficient in themselves to wipe out the whole $2 500 and leave the system to the good, as an operating proposition, by the absence of transhipment costs and the restriction to the development of traffic resulting from delay, loss, and damage to goods at the transhipment junctions.

If the system in question had been isolated, the meter-gauge over the whole system would probably have been as advantageous as the broad-gauge over the whole system; nor is it improbable that a 4 ft. 8½-in. gauge, with standards of construction similar to those of the existing gauges, would have obtained similar results. However, the question of the extensions and their gauge did not arise until the broad-gauge section adjacent to the port had been in operation many years, and the acceptance of the meter-gauge for the whole system was impossible for the following reasons:

1.—The lines which now comprise the system had not been brought under one agency at the time the meter-gauge was commenced, and the then diverse interests rendered impossible a complete system on that gauge.

2.—The broad-gauge was in existence, and its conversion would have been an unjustifiable expense.

3.—The port authorities have always very rightly opposed the entrance of mixed gauges to their docks, jetties, and warehouses.

This case is in some respects similar to the problem the author describes in the Argentine, and the writer holds that the correct solution would have been extension by lines of light construction on the 5 ft. 6-in. gauge, and is so far in agreement with the author's solution for the Argentine alone, namely, gradual conversion to 5 ft. 6-in.

Mr. Osborne.

There is considerable difference, however, between the selection of gauge for new lines and the question of converting existing lines to that gauge. In the first case, the extra expense amounts to little; in the second, it is a very important factor, and there must be considerable saving in the cost of operation to justify it. To the writer the solution appears to be: first, the construction of all future lines to the 5 ft. 6-in. gauge, then the conversion to broad-gauge of the more important narrow-gauge main lines, and afterward the gradual conversion of the less important narrow-gauge branch lines.

The writer, however, is not in accord with the author in his selection of the 4 ft. 8½-in. gauge for all lines in South America other than existing broad-gauge lines in the Argentine, which are to be left as they are.

It is not conceivable to the writer that the efficient, important, and financially strong broad-gauge companies of the Argentine, bound together with mutually friendly interests, will consent to remain as they are, and inactive, while they are being isolated and attacked by the 4 ft. 8½-in. gauge, and thus forced to convert to that gauge. It is much more likely that they will successfully seek to preserve their integrity by extensions opening up new territory, and then, on the Argentine will be thrown all the disablements of a mixed gauge which, in the writer's experience, is much more detrimental to the working of a railroad system as an efficient transportation machine than is the mere question of gauge, provided the same gauge be adhered to throughout. If the possibility (which the writer doubts) of a single gauge for South America be admitted, it should be either 5 ft. 6-in. or 3 ft. 3⅜-in., as the majority of the existing mileage is of those gauges.

The author's selection of a 4 ft. 8½-in. gauge for the whole of South America appears to be a compromise between the existing broad and narrow gauges, and based largely on the assumptions that the results obtained on the 4 ft. 8½-in. gauge of the United States and Canada show their railway systems to be the most efficient in existence, and that this efficiency is due, largely, if not entirely, to their gauge.

The writer, being familiar with the working of two large systems: one a 5 ft. 6-in. system which carries 200 long tons of freight 1 mile at a charge of $1.00 and pays 10% on the investment, the other a meter-gauge system, with only one-sixth the freight traffic density of the former line, which carries 111 long tons of freight 1 mile at a charge of $1.00 and pays 8⅓% on the investment, is inclined to question the first assumption, and, as to the second, without disparagement to those of faith and vision who found in a railway of 4 ft. 8½-in. (the then narrow) gauge an instrument for opening up the interior of the North American continent, or to those whose energy and resource-

fulness have enabled this gauge to cope with present traffic conditions and give the results to-day obtained, the writer's experience is that, under conditions now obtaining in this portion of North America, the 4 ft. 8½-in. gauge is a mistake, and that if the gauge were 5 ft. 6-in. the country would now be better served, and for reasons similar to those given by the author in casting the narrow gauge in favor of a broader one. Mr. Osborne.

The forces which have impelled the 4 ft. 8½-in. gauge in North America to economize by increasing the train load and so allowing revenue-ton-miles to increase while reducing revenue-train-miles are world wide, and where the 5 ft. 6-in., meter and other gauges have a live management, they affect them in the same way as they have affected the North American 4 ft. 8½-in. gauge, and South America will gain nothing that she would not otherwise receive along these lines by tearing up her existing 5 ft. 6-in. and 3 ft. 3⅜-in. gauges to replace them by 4 ft. 8½-in.

If the conditions and density of traffic in the area under discussion (Bahia Blanca to north of Rio) are ever likely to approach the conditions and density of traffic of the railways of the United States and Canada, then the extension of the Argentine 5 ft. 6-in. is undoubtedly the gauge that should be made universal.

The writer doubts whether such will ever be the case. There may be considerable similarity between conditions in the southern part of the United States and the southern part of this area, but at their northern extremities one is in the Tropical and the other is in the North Temperate Zone. So far as the writer's experience goes, the inhabitants of a tropical climate do not develop the vigor and capacity for intense effort that lead one to suppose they will ever develop a commerce sufficient to require and justify the railway facilities given in Northern Europe and the northern part of North America. The development of the meter-gauge in the tropical and the broad-gauge in the temperate portion of the area under consideration is not improbably an adjustment to the economic needs of each area, and the writer is inclined to look for the economic and economical development of these areas and their railways along the existing majority gauges, and would endeavor to consolidate the northern lines on the meter- and the southern lines, with the Argentine lines, on the broad-gauge, the dividing line between the gauges, in the absence of political and military considerations, being the traffic-shed between areas tributary to the ports of Buenos Aires and Rio, across which but little traffic would pass.

The writer's experience is that all gauges are good and capable of efficient operation where the standard to which the line is constructed is suitable to the character of the traffic to be moved, but that mixed

Mr. Osborne. gauges and breaks of gauge are an unmitigated evil. In the situation disclosed by the author, the needs would be met by preserving both the 5 ft. 6-in. and 3 ft. 3⅜-in. gauges, which comprise about 80% of the existing railway mileage, as each gauge could be confined to a definite area where the disabilities which arise from a break of gauge on a line of large through traffic would not occur.

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THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

Discussion.*

BY MESSRS. M. SÉJOURNÉ AND GEORGE L. NORRIS.

Mr. Séjourné.

M. SÉJOURNÉ, ESQ.† (by letter).—The writer takes great interest in the problem of the use of steels of high elastic limit for the construction of bridges, and would second the author's efforts to obtain in France—at least for large works—the use of a different kind of steel from that which is now specified, and having the following characteristics:‡

$R = 42$ kg. per sq. mm.

$E = 24$ kg. per sq. mm.

$A = 25\%$ in a standard length, $L = \sqrt{66.67\ S}$

In France it is not usual to construct such great bridges as in America; consequently, the use of steels of high resistance is not as necessary, and the French steel works hesitate to supply themselves with tools for producing in great quantities steels which they are not sure of selling.

The writer is at present engaged in designing a large metal bridge over the Durance, the longest span being 121 m. (397 ft.). On account of the importance of this work, one of the largest steel works in France was requested to quote prices for furnishing carbon steel having the following characteristics:

$R > 50$ kg. $\quad E > 27$ kg. $\quad A = 20$ per cent.

The steel works replied that it would agree to these specifications, provided that an increase in price of 1 Franc per 100 kg. more than for ordinary 42-kg. steel was allowed, but without impact test.

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* Continued from August, 1914, *Proceedings*.

† Ingénieur en Chef du Service, Chemins de Fer de Paris à Lyon et à la Mediterranée.

‡ It will be noted that these characteristics differ from those given to the author and reproduced by him in his article in *Le Génie Civil*, No. 15, August 7th, 1909.

Mr. Séjourné. In using steels of high elastic limit it seems to the writer to be indispensable to have all the necessary guaranties with reference to its brittleness, and to have recourse, for that, to methods even more stringent than those now used for 42-kg. steel.*

Although French steel works allow tests for brittleness for high-priced special steels, they are generally opposed to them for steels of ordinary make designed for bridges and metal frameworks. These conditions will perhaps be bettered, thanks to developments in the manufacture of steel due to the electric furnace, which produces a material that is not brittle because it is pure. Unfortunately, its price is still too high.

The writer commends the author's endeavors to discover a constituent alloy of great tensile strength, high elastic limit, and of reasonable price. The problem is doubtless difficult to solve, but it is not beyond the science and ingenuity of American engineers, and it is hoped that the author's researches and tests may result in the desired metal.

Mr. Norris. GEORGE L. NORRIS, ESQ.† (by letter).—The author has shown the great advantages to be derived in bridge construction, especially for long spans, by the use of steel with high elastic limit. To obtain the high elastic limits which he mentions—60 000 to 100 000 lb. per sq. in.—it will be necessary to use alloy steels.

With alloy steel it is entirely feasible to obtain elastic limits of from 90 000 to 100 000 lb. by heat treatment, and even also in the condition as rolled, but this, of necessity, would increase considerably the difficulties of shop manipulation. In the condition as rolled, the alloy steels would naturally be too variable in hardness for safe use.

In the case of nickel steel, in order to attain an elastic limit of 60 000 lb. or more, it is necessary to increase the percentage of carbon to undesirable limits.

To produce steel having an elastic limit approaching 100 000 lb., and workable under ordinary shop manipulations, the writer believes that recourse will be had to the use of vanadium alone, or with some other metal such as chromium in the steel. Vanadium is undoubtedly the element which, together with carbon, acts with the greatest intensity in improving alloys of iron, that is, in very small percentages. The quantity of vanadium generally alloyed with steel, excepting tool steels, is about 0.15 or 0.20 per cent. The price of vanadium has been reduced 60% within the past 3 years, and it is now low enough to

* In reference to the information given to the author, and reproduced by him in his article in *Le Génie Civil* (August 7th, 1909), it is said that the specifications of the French railroads "do not require impact tests, which is greatly to be regretted." This is true, but the writer does not make use of these specifications in his office, and for 12 years he has required such tests on all rolled steel.

† Metallurgical Engr., American Vanadium Co., Pittsburgh, Pa.

bring it within consideration for use in steel for eye-bars and other parts of long-span bridges. Mr. Norris.

Vanadium has generally been used in combination with chromium, although when used with nickel or nickel and chromium it gives greatly increased physical properties; these latter combinations are naturally more expensive. Added to simple carbon steel, especially if the percentage of manganese is more than 0.60%, it gives an increase of about 40% in the elastic limit and about 20% in the tensile strength, with practically the same or even greater ductility.

What is known as Type "A" chrome vanadium steel will fulfill the requirements of high elastic limit and be workable under shop manipulations. The cost of this steel would be only slightly in excess of that of 3½% nickel steel. Its chemical range would be:

Carbon	0.17 to 0.27	per cent.
Manganese	0.40 to 0.60	" "
Chromium	0.60 to 0.90	" "
Phosphorus, not more than	0.05	" "
Sulphur, not more than	0.05	" "
Vanadium, not less than	0.15	" "

As rolled, in plates, shapes, and bars, this steel would have approximately the following properties:

Elastic limit	Tensile strength.	Elongation in 8 in.	Reduction of area.
60 000	85 000	12%	45%
to	to	to	to
80 000 lb.	110 000 lb.	20%	60%

By the simple operation of heating to 1 100 or 1 150° Fahr., any irregularities in hardness and strength, due to variation in rolling temperatures or uneven cooling, can be removed without appreciably decreasing the elastic limit and tensile strength, and, at the same time, increasing the ductility. This operation would not be an expensive one, and experience might show that it would be unnecessary for this grade of steel.

Type "A" chrome vanadium steel can be sheared, punched, reamed, bent, etc., without any considerable increase of shop manipulations.

Much higher elastic limit and tensile strength can be obtained from this steel by heat treatment, quenching and tempering, but this would increase the shop manipulations very materially, necessitating drilling instead of punching and reaming. It would also add materially to the cost. Any bends would have to be made before heat treatment.

Rather than heat-treat steel of this type, it would be preferable to attain elastic limits of 80 000 to 100 000 lb. by increasing the carbon percentage, and normalize the steel by heating to 1 100 or 1 150° Fahr.

Mr. Norris.

The range in chemical composition for this grade would be:

Carbon	0.30	to 0.40	per cent.
Manganese	0.40	to 0.60	" "
Chromium	0.60	to 0.90	" "
Phosphorus, not more than	0.05	"	"
Sulphur, not more than	0.05	"	"
Vanadium, not less than	0.15	"	"

Plates $\frac{1}{2}$ in. thick, in the upper range of this composition, have shown the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 2 in.	Reduction of area.
125 000 lb.	145 000 lb.	18%	58%

These plates could be punched and sheared as rolled, but it would be advisable no doubt to normalize, as before described, and also to drill rather than to punch the holes.

Plates $\frac{5}{8}$ in. thick, at the low range of this composition, have shown the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 2 in.	Reduction of area.
83 000 lb.	116 000 lb.	20%	45%

Normalized, this grade of steel should give, in plates and shapes, the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
75 000 to 100 000 lb.	100 000 to 125 000 lb.	More than 12%	More than 50%

The cost of these chrome vanadium steels would be about 3 cents per lb. more than for ordinary carbon steel.

The writer believes that a simple carbon vanadium steel would prove commercially more attractive than the vanadium steels containing chromium or nickel or both. It is possible to obtain a very material increase in elastic limit by the addition of vanadium to a simple carbon steel, and such steels can be manipulated almost, if not quite, as readily as ordinary carbon steel.

Tests from $1\frac{1}{16}$-in., round, rolled bars of acid open-hearth casting steel of the following composition give a very good idea of what can be expected from this type of steel and how it compares with simple carbon acid open-hearth casting steel:

Carbon	0.26	to 0.27	per cent.
Manganese	0.60	to 0.64	" "
Silicon	0.25	to 0.25	" "
Vanadium	0.21	to 0.00	" "
Phosphorus, less than	0.05	to 0.05	" "
Sulphur, less than	0.05	to 0.05	" "

Mr. Norris.

Elastic limit	70 000	to 56 000	lb.
Tensile strength	88 000	to 72 000	"
Elongation in 2 in.	28.5	to 34	per cent.
Reduction of area	57.5	to 58	" "

From these and other tests, it would seem feasible to specify as follows, for this type of steel:

Carbon	0.20	to 0.30	per cent.
Manganese	0.60	to 0.80	" "
Phosphorus, not more than		0.05	" "
Sulphur, not more than		0.05	" "
Vanadium, not less than		0.15	" "

Elastic limit	50 000	to 70 000	lb.
Tensile strength	80 000	to 100 000	"
Elongation in 2 in., more than		20	per cent.
Reduction of area, more than		45	" "

The additional cost of vanadium steel of this grade over that of ordinary carbon steel should not be more than 1 cent per lb., and very likely would be less.

For built-up members of vanadium steel, the writer believes that vanadium steel rivets should be used, in order to utilize more fully than would be otherwise possible the high physical qualities of the vanadium steel shapes and plates. The rivets could be either of chrome vanadium steel or simple carbon vanadium steel. The composition for chrome vanadium steel rivets should be:

Carbon	0.15	to 0.20	per cent.
Manganese	0.30	to 0.50	" "
Chromium	0.40	to 0.60	" "
Phosphorus, not more than		0.05	" "
Sulphur, not more than		0.05	" "
Vanadium, not less than		0.15	" "

This steel, as rolled in rounds, would have the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
50 000 to 65 000 lb.	70 000 to 90 000 lb.	More than 18%	More than 50%

Single and double shear tests of rivets of steel of this type show:

For single shear		20	per cent.
For double shear		30	" "

greater than for ordinary rivets with a tensile strength of 55 000 lb.

Mr. Norris.

The simple carbon vanadium steel should have a chemical range of:

Carbon	0.15 to 0.20 per cent.
Manganese	0.60 to 0.80 " "
Phosphorus, not more than	0.05 " "
Sulphur, not more than	0.05 " "
Vanadium, not less than	0.15 " "

This steel, as rolled in rounds, would have the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
40 000 to 55 000 lb.	65 000 to 85 000 lb.	More than 18%	More than 50%

There should be no difficulty in driving rivets of either of these types of steel.

In the case of eye-bars, possibly the conditions are more favorable for the use of alloy steels than for built-up members. They can be more readily and advantageously heat-treated to develop high elastic limits.

In 1909 a number of tests were made of full-sized eye-bars, heat-treated, of chrome vanadium and chrome nickel vanadium steel. These eye-bars were made from bars 14 by 2 in., had 34-in. heads with 12-in. pin-holes, and were 25½ ft. long over the pin-holes.

The chrome vanadium steel bars experimented with were too high in carbon, and the results obtained for elongation in 20 ft. were not quite as good as in the case of the bars from the chrome nickel vanadium steel. Tests from this latter steel gave results ranging as follows, depending on the drawback or annealing temperature after quenching:

	No. 1.	No. 2.
Elastic limit	63 280 lb.	80 480 lb.
Tensile strength	93 500 "	99 800 "
Elongation, 12 in.	35 per cent.	32.5 per cent.
Elongation, 20 ft.	14.2 " "	7.9 " "
Reduction of area	50.8 " "	52.3 " "

Tests on 2 by ½-in. specimens turned up from the disks cut out in machining the eyes check the elastic limit and the tensile strength obtained from the full-sized bar very well. A test from the eye disk of Bar No. 2 showed:

Elastic limit	83 040 lb.
Tensile strength	94 140 "
Elongation, 2 in.	25 per cent.
Reduction of area	71.9 " "

Mr. Norris.

The chemical composition of this steel was approximately:

Carbon	0.25 per cent.
Chromium	0.90 " "
Nickel	1.20 " "
Vanadium	0.17 " "

Based on these tests, it is perfectly feasible to specify as follows for heat-treated eye-bars of chrome vanadium or chrome nickel vanadium steel:

Elastic limit	65 000 to 80 000 lb.
Tensile strength	85 000 to 105 000 "
Elongation in 2 in.	20 per cent.
Reduction of area	50 " "

The cost of heat treatment for eye-bars would probably be from ¼ to ⅜ cents per lb. The cost per pound of the chrome vanadium or chrome nickel vanadium steel would be about 3 cents more than that for carbon steel.

The writer believes that eye-bars made from simple carbon vanadium steel of the following composition, either heat-treated, quenched and annealed, or normalized, heating to about 1 100 or 1 150° Fahr., after the heads have been forged, will give elastic limits approaching those of the more expensive chrome or chrome nickel vanadium steels:

Heat-treated or normalized:
Chemical composition.

Carbon	0.30 to 0.40 per cent.
Manganese	0.60 to 0.80 " "
Phosphorus, not more than	0.05 " "
Sulphur, " " "	0.05 " "
Vanadium, not less than	0.15 " "

Physical properties.

Elastic limit	60 000 to 75 000 lb.
Tensile strength	85 000 to 100 000 "
Elongation in 2 in., more than	18 per cent.
Reduction of area, more than	45 " "

The writer has given considerable attention to simple carbon vanadium steel containing from 0.60 to 0.80% of manganese. This steel will be the commercial or every-day vanadium steel of the immediate future. It is much cheaper than the vanadium steels containing chrome or nickel. It is at least 40% better than simple carbon steel of otherwise the same composition, and is bound to be used extensively in the near future for rails, general and locomotive forgings, and special structural purposes. It presents no especial difficulties in manufacture over simple steel.

Mr. Norris.

The quantity of vanadium which remains in the steel is about 80% of that added. It is very evenly distributed. No instances of segregation are known, and it has a strong influence in overcoming the segregation of other elements, particularly carbon.

Vanadium will readily alloy with nickel, and better results can be obtained from a 2% nickel vanadium steel than from a 3½% straight nickel steel of the same carbon content.

There would be absolutely no advantage gained in the use of titanium with vanadium. Vanadium is an alloying metal and is used as such, not as a scavenger or deoxidizer. It is only the vanadium which alloys with the steel that can be considered as influencing the quality of the latter. Titanium has a little merit as a deoxidizer over silicon or aluminum, and its action is apparently a surface reaction. It is a question whether the observable reaction, when evident, is not with the highly oxidizable basic slag with which it comes in contact, or even with the atmosphere.

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THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE.

Discussion.*

BY MESSRS. O. E. MEINZER AND KENNETH ALLEN.

Mr. Meinzer.

O. E. MEINZER, ESQ.† (by letter).—Mr. Lee's investigation in Owens Valley is a valuable contribution to the study of ground-water. The demand for irrigation supplies and the increasing availability of ground-water because of improved irrigation and cultural methods and decreased pumping costs have created a need for information as to the magnitude of ground-water supplies, the question being primarily not as to the quantity stored in the earth but as to the annual recharge or safe yield. Any contribution to the methods for estimating the annual supplies of ground-water, therefore, is especially valuable at this time. Four principal methods or groups of methods, which may be called the percolation, underflow, water-level, and evaporation methods, have been used. The first consists in estimating the quantity of water that percolates into an underground reservoir from streams or other surface sources; the second in measuring the flow of ground-water at selected cross-sections, being similar in principle to the gauging of surface streams; the third in observing fluctuations in the water-table, which represent filling or emptying of the underground reservoir; and the fourth in measuring the discharge of ground-water through evaporation from soil and plants. All these methods are laborious and difficult to apply, and none of them can be expected to produce precise results, but they are valuable, nevertheless, because they give some tangible basis for estimating ground-water supplies.

* Continued from August, 1914, *Proceedings*.

† In Charge, Ground-Water Div., Water Resources Branch, U. S. Geological Survey.

Mr. Meinzer.

In the Owens Valley investigation, Mr. Lee used both percolation and evaporation methods, his distinctive contribution consisting in developing the latter method and placing it on a quantitative basis.

The evaporation method gives promise of extensive applicability because a large number of the ground-water reservoirs that will be developed for irrigation, discharge wholly or chiefly by evaporation. Débris-filled basins are the most characteristic features of the United States west of the Rocky Mountains. They are of two types: Those which discharge ground-water through springs and evaporation areas, as described by Mr. Lee, and those which do not. In the geologic literature dealing with these basins this fundamental distinction is generally ignored, and the characteristics of the evaporation areas are commonly accounted for by the wholly inadequate explanation of evaporation of surface waters. The process of ground-water evaporation, however, has been clearly stated by F. H. Newell,* M. Am. Soc. C. E., in one of the earliest papers on water resources published by the United States Geological Survey, and in recent ground-water investigations the significance of evaporation areas has come to be clearly recognized. Mr. Lee has furnished experimental data showing that these areas are quantitatively important in discharging ground-water.

Work in numerous débris-filled basins has shown that it is entirely feasible to ascertain from surface indications whether or not a basin is discharging ground-water by evaporation. The evaporation areas in some of the basins have been mapped with nearly the same accuracy that is possible in mapping geologic formations. The three criteria, all of which are suggested in the paper, are (1) moisture of the soil; (2) soluble salts at the surface; and (3) native plants that feed on ground-water. Experience is necessary, of course, for a proper application of these criteria, but they are trustworthy when rightly used. Moreover, they can be tested at any time by making a shallow boring, for ground-water evaporation takes place only where the water-table is near the surface. In a recent ground-water survey of the Big Smoky Valley, Nevada, the relations between the vegetation zones and the depth to the water-table were found to be very similar to those stated by Mr. Lee (page 867),† but the plant species that serve as indicators of ground-water are not the same in different parts of the West. The mapping of evaporation areas is important not only because these areas make possible estimates of the annual supplies, but also because they reveal the base level of the ground-water surface, and thus make possible a forecast of the depth to water in other parts of the basins in which they occur.

Mr. Lee's first conclusion (page 818)† is open to criticism in being too general. On the basis of his investigation in the Owens Valley

* "Water Supply for Irrigation," U. S. Geol. Survey, 13th Annual Rept., Pt. 3, p. 29, 1893

† *Proceedings*, Am. Soc. C. E., for April, 1914.

Mr. Meinzer.

he concludes that the underground reservoirs of California and the Southwest are water-tight rock basins. Many of the débris-filled basins of the Southwest, however, even those comparatively well enclosed by mountains, have no ground-water discharge through springs, evaporation, or transpiration, and it must be assumed that the supplies which they undoubtedly receive are disposed of entirely through rock absorption or underground channels of escape. Some of the reservoirs which discharge water by springs, soil evaporation, and transpiration, no doubt also suffer losses through rock absorption or leakage. In this respect each basin forms a separate problem, the amount of underground loss depending on the stratigraphy and structure as well as the topography. It should be noted that such loss, if heavy, will make the estimates obtained by the percolation method too large and may make those obtained by the evaporation method too small. One of the sources of the public supply of Goldfield, Nev., has consisted of wells in a closed débris-filled basin which has no discharge by evaporation. Such a basin will yield some water even though it has no natural discharge through springs, evaporation, and transpiration, but its yield will be less than the quantity that percolates from surface sources to the water-table.

The writer was disappointed in not finding in Mr. Lee's paper a more definite discussion of the probable percentage of accuracy of his results, as such a discussion would have added greatly to the value of the paper. Mr. Lee's analyses both of recharge and of loss are excellent, but in order to reach quantitative conclusions he was obliged to make numerous assumptions in respect to both. Although these assumptions were, the writer believes, made carefully and with good judgment, they must have introduced considerable errors into the computations. As assumptions enter into both estimates, neither one can be regarded as a check on the other. The fact that the two estimates are of the same magnitude justifies added confidence in the general results, but the fact that they differ by less than 4% does not indicate, of course, that the percentage of error is within 4%, and can hardly be considered a confirmation of the reliability of the data and the correctness of the assumptions in either computation (page 881)*.

The four sources of percolation are given as (1) direct precipitation; (2) stream flow; (3) irrigation; and (4) flood-water in the valley floor. It is assumed that the high mountain areas shed all precipitation except that which is lost by evaporation; that on the more elevated parts of the intermediate mountain slope 75% of the precipitation, and on the lower parts 70, 65, and 60%, respectively, join the underground supply; that the contributions to the underground supply on the outwash slopes range, according to zones, from 0 to 60%, making an average of 16%; and that the contribution on the valley floor is 4 sec-ft. All these assumptions are based on care-

*Proceedings, Am. Soc. C. E., for April, 1914.

Mr. Meinzer.

ful, general observations, but are of course only approximations and liable to large errors. Moreover, assumptions are involved as to the areas belonging to the various zones and the amount of precipitation in each (page 875)*. Several assumptions are also made in the estimate of recharge from stream channels, in the conclusion that 50% of the irrigation water is added to the underground supply, and in the estimate of the contributions made by the flood-waters.

The writer agrees with Mr. Lee that the discharge from the underground reservoir can be determined more accurately than the percolation into it, but the methods of estimating discharge also involve a number of elements of uncertainty. If the writer understands rightly the author's discussion of the discharge from irrigated land, the 50%, or 18 sec-ft., of irrigation water discharged by evaporation and transpiration (page 872)* is the portion that was not added to the underground supply (page 881)* and, therefore, should not be included with the discharge from this supply.

The largest element in ground-water discharge is that of evaporation and transpiration from uncultivated land. It is on this question that Mr. Lee has made his most important contribution, and that part of his paper dealing with this phase of the subject deserves, therefore, the most critical consideration. Some of the factors that enter into this estimate were determined by experiment and others by field survey. Experimental errors were involved in the difference between natural and artificially packed soils, in the difference between the vegetation in Nature and in soil tanks (pages 855, 856, 861, and 862),* in uncertainties as to the actual water-level in the tanks (pages 852 and 853),* and in the unavoidable fluctuations in the water-levels in each tank (Tables 9 to 14, inclusive). These experimental errors are represented in part only in the diagrams in Fig. 10. In the summer diagram, which is the more important one, the results from Tanks Nos. 5, 6, and 7 fall nearly on the curve used by Mr. Lee in his calculations, the result from Tank No. 4 being more than 20% too low, that of Tank No. 3 more than 20% too high, and that of Tank No. 2 nearly 40% too low. It should be noted, however, that although these results involve large experimental errors, they corroborate each other in a general way and are of great value in furnishing reliable data on the general magnitude of one of the most important processes in ground-water circulation.

In applying the experimental data to the basin under investigation, inaccuracies were involved in the sizes of the areas having specified depths to the water-table, in the fluctuations of the water-table, in difference in the range and rate of capillary rise due to difference in the character of the soil, and in difference in the density of vegetation

* *Proceedings*, Am. Soc. C. E., April, 1914.

and kinds of plants that draw water from the underground reservoir. An error was also involved in the fact that observations covering only 1, 2, or 3 years did not give average evaporation conditions, just as precipitation and stream-gauging data for a period of the same length do not afford reliable averages. With 142 observation wells on the valley floor (page 866),* the error as to the water-table cannot have been large, yet the rate of discharge varies so greatly with small changes in depth of the water-table that even slight inaccuracies in determining one produce appreciable errors in calculating the other. Although the valley floor in that part of Owens Valley investigated by Mr. Lee has relatively uniform conditions, there is luxuriant salt grass in some parts and an entire absence of vegetation in others (page 828),* whereas the experiments did not cover these different conditions. Moreover, no account was taken of the zone having depths to the water-table between 8 and 12 ft., although the greasewood and rabbit brush in this zone probably draw water from the underground reservoir (page 867).* Mr. Meinzer.

The writer agrees with Mr. Lee that the evaporation method is the most feasible one for estimating ground-water recharge in many of the débris-filled basins, especially where there are as yet few developments, but its application is far from being a simple matter. The data which he obtained in Owens Valley have value in making rough estimates of annual recharge in valleys in which the evaporation areas are mapped and reliable observations are made as to the character of the soil and vegetation and the distance to the water-table in the different zones of such areas, but to assure any considerable degree of accuracy for most valleys it will be necessary not only to sink a large number of observation wells and to keep them under observation for one or more years, as was done in Owens Valley, but also to obtain a great deal more information as to the rate of discharge under various conditions of soil and vegetation. The conditions in the evaporation areas of most valleys are far from uniform, the soil ranging from dense clay to coarse sand or gravel, and the vegetation embracing a number of diverse species and being entirely absent over large tracts.

The writer wishes to urge the importance of further investigations along the lines suggested. The lowest parts of many of the closed basins are underlaid by clay cores destitute of vegetation but surrounded by zones of less dense soil in which evaporation and transpiration are active. The dissemination of the soluble salts instead of their concentration at the surface and other conditions lead him to believe that on these clay cores the ground-water discharge is sluggish, but definite tests are needed as to the quantity of discharge and its relation to the distribution of the soluble salts. Among the common native plants (besides the salt grasses) which apparently discharge ground-water, are samphire (*Spirostachys occidentalis*), iodine weed (*Suaeda*),

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Mr. Meinzer.

alkaline sacaton (*Sporobolus airoides*), certain species of salt bush (*Atriplex*), big greasewood (*Sarcobatus vermiculatus*), rabbit brush (*Chrysothamnus graveolens*), buffalo berry bush (*Shepherdia*), and mesquite (*Prosopis*). Mesquite does not thrive in the shallow-water areas where the soil is dense and alkaline, but is often dominant in a zone of moderate depth to water surrounding a shallow-water area. It is important to know whether the mesquite actually feeds on ground-water, and if so, from what depth and at what rate it is able to lift this water to the surface.

Mr. Lee's final conclusions (page 887)* sum up admirably the esssential factors in the problem of safe yield. As it is not generally practicable to draw any large part of the ground-water of one segment of a valley to another, a proper distribution of wells is necessary in order to reduce the residual losses to the lowest possible quantity. Overdraft with serious lowering of the water-table may occur in certain segments while evaporation of ground-water contributed by other segments is still in progress on the lowest lands. Such loss can be prevented only by increased withdrawals in the undeveloped segments, and if these segments have little or no land that can be profitably irrigated, the loss may be unavoidable.

Mr. Allen.

KENNETH ALLEN, M. AM. SOC. C. E. (by letter).—Underground water supplies, as well as surface supplies, depend on the rainfall, the catchment area, and the available storage, with its accompanying losses by overflow, leakage, and evaporation; but the problem for the engineer is more difficult in the case of underground supplies, as the true limits of the catchment area, the storage capacity, and the probable amount of the losses mentioned can only be determined approximately after a pretty thorough examination of the sub-surface conditions, that is, the configuration of the permeable strata, their impermeable confines, and their physical characteristics—permeability, etc. In the majority of cases much of this information is inaccessible, and more or less dependence is placed on such collateral evidence as the yield of neighboring wells and the results obtained by sinking test wells.

The limitations imposed by enclosing impervious formations and by sub-surface evaporation are clearly and interestingly presented by the author, and their importance is shown under the conditions discussed. In most well developments, however, evaporation is of minor significance. This is not only on account of greater humidity, but because most supplies percolate through an unenclosed stratum for perhaps many miles and because evaporation from this stratum is usually prevented by the superposition of one or more impervious strata.

* *Proceedings*, Am. Soc. C. E., for April, 1914.

Mr. Allen.

In practice the available yield is subject to further limitations. The water-bearing stratum may consist of an impervious rock containing crevices or fissures due to movements in the earth's crust or water-courses caused by the solvent action of the water itself. The latter are of frequent occurrence in limestone and chalk formations and the former in sandstones and the denser igneous rocks. Fissures occur more frequently near the surface than at great depths, and, as the cost of drilling increases with the depth, deep borings in search of water-bearing fissures are not often profitable. The same money can be spent to better advantage in making several test borings at moderate depth. Although the results of such borings are uncertain, supplies, when obtained from fissures, are often abundant and the wells free from the clogging so often experienced with sand. The yield of a well in sand is directly limited by the porosity of the latter and the available head. In fine sands there is a large loss of head due to percolation near the strainer in order to maintain the necessary flow. This difficulty may be overcome by increasing the length of the strainer if this does not exceed the thickness of the water-bearing stratum, but this is at the expense of a greater first cost as well as an increase in lift due to the fineness of the material. In such cases the loss of head is often reduced by removing the sand about the strainer and filling the pocket with gravel, or else by substituting a larger number of driven well points from 2 to 3 in. in diameter for the large (6 to 12-in.) casing and strainer.

In those cases where the water occurs in a stratum of fine sand of small thickness, the difficulty is further increased. The writer tested a stratum of this kind several years ago with the view of developing a supply of some 5 000 000 gal. daily for a Southern city. Water was found in apparent abundance throughout a large area at a moderate depth, the land was low, covered with forest, not far from a stream of considerable size, and on it were several large springs. On sinking numerous test wells on a line about 4 miles long, however, it was shown that the water-bearing stratum, besides being of a fine sand, was so thin that the cost of developing the desired supply would be prohibitive.

Water supplies found in deep deposits of sand or drift, especially those derived from distant and extensive catchment areas, such as exist south of the great terminal moraine on the south side of Long Island and on the Atlantic Coastal plain from Sandy Hook to Florida, are most favorable for development. Well-known examples are the 1 400-ft. Ponce de Leon well at St. Augustine, said to furnish 10 000 000 gal. daily, and the 1 970-ft. well at Charleston furnishing 1 250 000 gal. daily. On the other hand, a well was bored in 1900-01 at Atlantic City, N. J., to a depth of 2 306 ft. (below the floor of Young's Pier), and although a good supply was found between depths of 780 and 860

Mr. Allen.

ft., in a stratum of sand tapped by numerous wells in the vicinity, the supply sought at the greater depth was not realized, and the well, then the deepest but one along the Atlantic Coast, was abandoned.

About 5 miles inland, the Water Department had a small collecting basin near its pumping station (Fig. 15), in which were driven a number of 4-in. tubes tapping a water-bearing stratum 24 ft. below. Near the basin was a 10-in. well (No. 3) extending to a depth of about 100 ft. to another water-bearing stratum which it was proposed to develop as an additional supply to the extent of 3 000 000 or 4 000 000 gal. daily. The capacity of this well was first tested by erecting over it an old 1 000 000-gal. Worthington pump, which the Department had, and connecting it with a steam line to the boiler plant at the station. During a 6-hour test with a 19-in. vacuum on the suction,

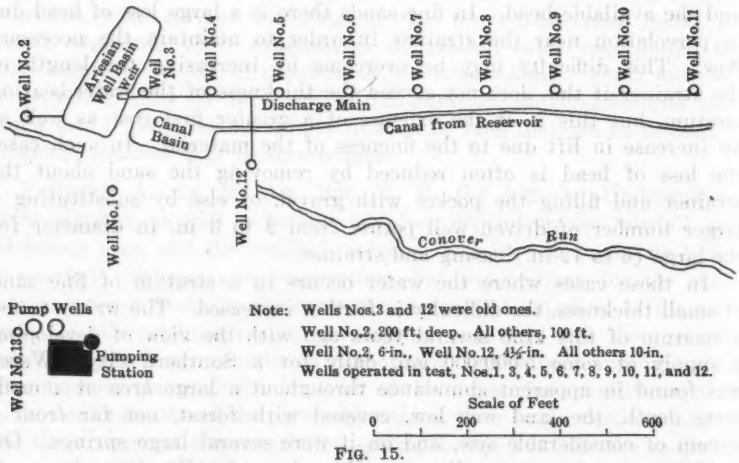

FIG. 15.

the delivery was practically uniform at a rate of 840 000 gal. per day, and as no downward flow was observed in the small tubes in the basin, it was concluded that there was no connection between these two strata. On the strength of this test, ten 10-in. wells, 125 ft. apart, were sunk to a depth of about 100 ft. and a 6-in. well (No. 2) to a still deeper stratum, 200 ft. below the surface. Besides these there were two old wells—the 10-in. well tested with the steam pump and a 4½-in. well (No. 12), both of which were carried to the 100-ft. stratum. The deeper 6-in. well flowed freely at a rate of 66 000 gal. per day and, by use of the air lift, at a rate of 400 000 gal. per day, or about 280 gal. per min. The 10-in. wells on short separate tests produced from 150 to 400 gal. per min. Ten of the 10-in. wells (No. 1 and Nos. 3-11) and the 4½-in. well (No. 12) were then connected with

Mr. Allen.

the compressor and given a 6-hour test. These wells together delivered 846 879 gal., or at a rate of 3 705 492 gal. per day. This was an average of 336 866 gal. per well, or 60% less than the delivery from Well No. 3 when operated alone. Moreover, the average lift in this well during the test was 24.56 ft., though the vacuum gauge on the suction during the earlier separate test indicated a lift of 21 ft. to about the same level. Assuming this well to have produced one-eleventh of the total quantity during the combined test, the effect of interference from the other wells, while operating all eleven at this rate, was to increase the lift by about 3.6 ft. while reducing the output by 60 per cent.

The yield under conditions found along the Coastal Plain region bears little relation to that from underground reservoirs of the closed-basin type described by the author, but somewhat similar conditions are met in the basin of the prehistoric Lake Passaic of Northern New Jersey. Wells in the outlet of this lake, now filled with glacial drift, furnish the water supply for East Orange. When first sunk, several of these 6-in. and 8-in. wells had a natural flow of from 400 to 500 gal. per min.

Above this outlet water-bearing sandstones underlie the ancient lake, but percolation to these beds is cut off on the northwest by the trap dikes forming the Orange or Wachung Mountains and on the southeast by the Palisades of the Hudson. Assuming a general southerly flow, percolation from a distance, therefore, is limited to that from the northeast, the limit in this direction, it is believed, being unknown.

These sandstones, nevertheless, furnish a large number of excellent well supplies in the vicinity of Newark and Paterson, chiefly by means of their numerous fissures.*

Data concerning a large number of these wells, collected by the writer a few years ago, indicate a great variation in the yield. They are commonly from 100 to 500 ft. in depth, and 6 or 8 in. in diameter, and generally furnish from 20 to 200 gal. per min. each, although failures are not infrequent, and several produce as much as 500 gal. per min.

* Report, State Geologist of New Jersey, 1903, p. 79; also Water Supply and Irrigation Papers, U. S. Geological Survey, No. 114, p. 96.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

Discussion.*

By E. G. Walker, Assoc. M. Am. Soc. C. E.

Mr. Walker.

E. G. Walker, Assoc. M. Am. Soc. C. E. (by letter).—At first thought it might appear remarkable that so little progress relatively has been made in the application of reinforced concrete to marine works in the United States; and in certain places in his paper, the author seems to attribute the present state of affairs to a conservatism of ideas on the subject utterly opposed to the general conception of the American progressive spirit. The writer is inclined to differ from this deduction, because he believes that in the United States as elsewhere the form of construction adopted for marine works is the result of economical evolution. A paper† on the piers of New York Harbor, recently presented before the Society, is an admirable illustration of this point, showing as it does in a very conclusive manner that special conditions evolve special types. A consideration of this paper leaves little doubt that the engineers of the Department of Docks and Ferries have arrived, by the judicious use of timber in some parts and reinforced concrete in others, at a type of pier of maximum economy under the conditions prevailing in New York Harbor. These conditions, however, are not comparable with those obtaining in certain teredo-infested waters on the coast of Great Britain, for example, where pitch pine piles will be dangerously eaten away in a few years

* Continued from August, 1914, *Proceedings*.

† "Modern Pier Construction in New York Harbor," by Charles W. Staniford, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 503.

Mr. Walker.

and where even Australian hardwood piles have to be renewed at comparatively short intervals. Although there appear to have been a number of unfortunate experiences in the United States, there is no question that concrete and reinforced concrete can be used successfully for marine works, because engineers have had ample experience in the use of both materials; but there are many locations where it may be more economical to use timber, in spite of its shorter life and other disadvantages. At the present time there is a large quay under construction on the River Tyne, in England, forming the largest item in a $1 000 000 scheme, which will be built of timber throughout. In North America, where timber is so much more easily obtainable than in the United Kingdom, the writer thinks that it will be a long time before it is completely ousted by reinforced concrete.

In many cases in the United Kingdom the application of reinforced concrete to quay and jetty construction has been brought about by economical necessity rather than by ideas of progressiveness *per se*. Most of the waters of the British ports carry teredo, and in many cases no timber but greenheart or jarrah will withstand its ravages. The Hennebique system of "ferro-concrete" construction, introduced into England by the late M. Mouchel, was first applied to sea-works at Southampton and, about the same time, to a jetty in the River Thames, at Purfleet. Both these works are referred to in detail in the paper. The success which attended them probably contributed very materially to the later popularity of reinforced concrete.

In general, under average British conditions, greenheart construction is about as expensive as reinforced concrete, pitch pine being, of course, considerably cheaper than either. Hence, in the absence of special conditions (such, for example, as those at Port Talbot mentioned by the author), pitch pine may be used in places where the marine borers are not too active, whereas in other locations either hardwood or reinforced concrete may be used, according to the local balance of general economy. In connection with this, the proved low maintenance cost of concrete work is a most important factor, and it is significant that in most cases where concrete pile-work has been introduced, its use has been continued.

The remarkable resilience of concrete piles under driving has been a most important factor in the increase in their use, but the writer thinks that the particulars given by the author on page 967* are scarcely applicable to a discussion on pile-work. It is reasonable to suppose that a hollow post of the dimensions given, reinforced with a large number of small rods, will be much more flexible than an ordinary pile, and such is the case. The resilience of piles is best

* *Proceedings*, Am. Soc. C. E., for April, 1914.

Mr. Walker.

exemplified by the treatment that they are able to stand in ordinary use, and particularly in driving. In the early days, the interposition of a special dolly between the pile and the monkey was considered essential, but since it has been found that properly arranged spiral winding is able to withstand all the punishment that the pile-driver can give it, the dolly is no longer used. This, in itself, is quite sufficient proof of the resilience of reinforced concrete piles; and there is plenty of other evidence.

A point which has arisen and which has been much discussed more particularly in connection with marine work in England is the subject of the relative advantages of pre-moulded members and members formed *in situ*. It has been contended that the former method is the better one by reason of its being more convenient, but the writer believes that, except where the special conditions of the case preclude it, it is desirable to build the structure as monolithic as possible. No matter how carefully the work is carried out, it is hardly possible to unite satisfactorily hardened concrete several weeks old with the new concrete that must be put in to form the joint; nor can one get in built-up work the strength obtained by the continuity of the steel from one member to another in monolithic work. The use of additional binding steel at the joints does not fully compensate for this. Advocates of the built-up system put forward the claim that monolithic work may be considerably weakened by the wrong disposition of the reinforcing bars, and that, therefore, it requires considerable and constant supervision; but surely the increased relative importance of the field-work in the case of pre-moulded construction entails quite as much detailed supervision on the job.

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SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

Discussion.*

BY HERBERT E. BELLAMY, ASSOC. M. AM. SOC. C. E.

Mr. Bellamy.

HERBERT E. BELLAMY, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with particular interest, not only on account of his familiarity with the subject, having dealt with questions of riparian rights in England, and of his knowledge of the systems in vogue throughout Australia, but because of the comprehensive and lucid manner in which the author has dealt with the whole subject. Indeed, the paper embraces practically every phase relating to the betterment in the administration of streams, so that there is very little, if anything at all, to add. The preamble presents amending principles suggested for adoption impartially, and with which, in every case, the writer cordially agrees. The paper brings together, in a small compass, a great mass of information dealing with the application of the doctrine of riparian rights in many countries, which cannot fail to be of the greatest value in the discussion of the subject by engineers.

Some years ago, when the writer was investigating some so-called riparian rights in connection with a proposed new water supply for a town in England, he was greatly puzzled to know what these rights were. Decisions of the Courts did not help him, because, as the author remarks, if the doctrine of riparian rights were carried into effect rigidly, water could not be diverted from a stream in any way. Several decisions could be quoted to support this latter contention if necessary, but the following brief abstract from a well-known case

*This discussion (of the paper by Clarence T. Johnston, M. Am. Soc. C. E., published in May, 1914, *Proceedings*, and presented at the meeting of September 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Bellamy.

merely serves to show that something more tangible should be substituted for the absurd and diabolical doctrine, as it exists in many places to-day. In the case, Mason *v.* Hill, it was decided as follows:

"A riparian proprietor can have no larger right than he has by nature against those above and below him. Hence the right to have a stream to flow in its natural state without diminution or alteration is an incident to the property in the land through which it passes * * *."

The author points out that the first principle which should be recognized is that of public ownership of streams, lakes, and other bodies of surface water. The writer would also include "all underground supplies." This recognition is the fundamental principle of success of stream administration throughout the Australian States. In the writer's opinion, the second principle, *b*, which the author does not discuss at great length, is the positive safeguarding of the interests of the first user of the waters of a stream as against later comers, even including State or National rights.

As the author mentions that the modern system adopted by Australia merits study, the writer submits the following brief digest of the Water Rights Act of 1912 which is in force in the State of New South Wales.

The Water Rights Act came into force in 1896 and was consolidated in 1902. On November 26th, 1912, assent was given to an Act consolidating the separate Acts relating to Water Rights, Water and Drainage, Drainage Promotion, and Artesian Wells. This Act is administered by the Minister for Public Works. On the passing of the original Act, the right to the use and control of all streams, whether perennial or intermittent, flowing in a natural channel, through or past the land of two or more occupiers, and of all lakes, swamps, lagoons, or other sheets of still water, whether permanent or temporary, situate within or fronting the land of two or more occupiers, passed from the owners or occupiers of the said land and became vested in the Crown; subject, however, to the reservation that the said occupiers shall have the right to use the water on their frontage for domestic or stock purposes, or for gardens up to 5 acres in extent, without the necessity for obtaining a license for any work used solely for those purposes.

Rights granted under the Mining Acts on any public or private statute, are also preserved.

Section 8 provides for the rights of the Crown in respect of works, and Section 9 for the rights of the occupiers to which the Act applies.

The definition of "Work" is comprehensive, and includes any dam, lock, reservoir, weir, flume, race, channel (whether an artificial channel or a natural channel artificially improved), any cutting, tunnel, pipe, sewer, and any machinery and appliances; and "Work to which

this part extends" means work connected with any river or lake flowing through, or past, or situate within the land of two or more occupiers, or with any water flowing in, to, or from, or being in any river or lake flowing or situate as aforesaid, whether such work be for water conservation, irrigation, water supply, or drainage, and whether such work be constructed before or after the commencement of this Act. Mr. Bellamy.

Section 10, and succeeding sections, provide for the granting of licenses to private individuals. Any person being in actual occupation of the site of a proposed work may make application for a license. The application should be forwarded to the Minister for Public Works, who will then advertise its receipt, together with a date and place at which a public inquiry will be held as to the advisability of granting the application. These inquiries are usually held by the local Land Board, and any person affected by the application may attend and give evidence for or against the granting of the application. Any person aggrieved by the finding of the Board may appeal to the Land Appeal Court within 28 days of the date of such finding. The finding of the Board is published in the *Government Gazette,* and, after the expiration of 30 days from the date of publication, the Minister, where the Board recommends the granting of the application, and provided no appeal is pending, shall issue a license for such period, not exceeding 10 years, as he may think fit, subject only to such conditions, if any, as may be embodied in the Board's finding. The applicant is notified by the Minister of the term for which it is intended to grant a license, and the amount of the fee which must be paid before a license can be issued.

Section 13 of the Act provides for joint application by two or more occupiers, as, for instance, persons on opposite sides of a stream who desire to construct a dam.

Section 17 is very important, because it secures to the holder of a license quiet enjoyment, and the sole and exclusive use, of the licensed work as against all persons, including the Crown.

Section 18 provides for a penalty for any alteration of work during currency of a license.

Sections 19 and 20 provide for the carrying out of works by the Crown, the formation of "districts" benefited by such works, the assessment of charges in each and every case for benefits accruing to the properties within the "district", and the payment of charges by the persons benefited. The actual procedure is conducted as follows:

"The Governor notifies by proclamation in the *Government Gazette* a proposal for the construction of a dam, lock, weir, channel or drainage work, as the case may be, to be constructed by the Crown, together with an estimate of cost. This notification is followed by the gazettal

Mr. Bellamy.

of the boundaries of a 'district,' the land within which will, in the opinion of the Local Land Board, be benefited by the construction of the proposed work, and within which water or drainage charges may be levied. It is then necessary that a 'two-thirds majority' of the occupiers, who must also occupy an area exceeding two-thirds of the total area within the proclaimed district, petition the Land Board, on the proper form, to carry out the proposed work. In default of this petition the proposal lapses. On receipt of the petition, the Land Board may report to the Minister, recommending that the work be carried out, and after the expiration of thirty days from receipt of the Board's recommendation, the Minister may carry out the work with funds legally available. On completion of the work, the Land Board, by direction of the Minister, assesses the charges to be paid by each individual occupier, and the aggregate of these charges must not exceed six per cent. on the total cost of the work. The charges commence from the date of completion of the work, which must be advertised in the *Government Gazette*. The assessment takes place in open court, and the persons affected may attend and give evidence on their own behalf. On the petition of persons liable to pay one quarter of the total assessment, the Land Board must make a fresh assessment."

The remaining Sections, 21 to 27, refer to miscellaneous matters such as injuries to works; power of entry; obstructing persons in the performance of duties; recovery of fees, charges, and penalties; appeal; consolidated revenue; and the power to make regulations.

The regulations at present in force provide penalties for pollution, obstruction of or interference with the water in any stream or lake, and for causing trees, or débris of any kind to fall into any stream or lake, and also prescribes the various forms to be used under different sections of the Act.

From a perusal of these Acts,* it will be found that many of the principles advanced by the author have been provided for and are in successful operation in Australia.

The author certainly deserves the thanks of all water engineers for the exhaustive and thorough manner in which he has treated a most delicate and vexed subject.

* Copies of the Water Rights Acts at present in force in three of the Australian States, namely, Act No. 44, 1912, New South Wales; Act No. 2016, 1905, Victoria; and Act No. 25, 1910, Queensland; are on file in the Library of the Society.

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THE CONSTRUCTION OF THE KLONDIKE PIPE LINE.

Discussion.*

BY MESSRS. G. B. PILLSBURY AND WALTER S. WHEELER.

Mr. Pillsbury.

G. B. PILLSBURY, M. AM. SOC. C. E. (by letter).—The description in this paper of the frozen soil in the Klondike is most interesting, but should perhaps be supplemented in order to prevent a misconception of the difficulties and possibilities of earthwork in Alaska. The writer was engaged for several years in wagon-road construction throughout the occupied portion of the Territory, and acquired a fairly intimate acquaintance with its frozen ground.

In the interior of Alaska—the area north of the enclosing coastal range—the mean annual temperature is below the freezing point. As the temperature of the ground some feet below the surface must be approximately the mean of the surface temperature, it follows that this ground is, in general, permanently frozen. The underground drainage being thus sealed, the ground-water level is high, and the country is extremely swampy. Deposits of swamp muck, therefore, are very common in the valleys and elsewhere, and they sometimes contain lenses of solid ice several feet in thickness. As the author remarks, when the moss that covers this muck is stripped, its ice content melts under the summer sun. The writer has often seen a narrow ditch quickly enlarge into a wide and deep gully through this action. It is very obvious that if ice is built on, it must be kept frozen, and in order to keep it frozen, its protecting coat of moss must be retained.

* This discussion (of the paper by W. W. Edwards, Assoc. M. Am. Soc. C. E., published in May, 1914, *Proceedings*, and presented at the meeting of September 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Pillsbury.

It by no means follows, however, that all the valleys are covered with muck and ice, any more than all the valleys in more temperate zones are covered with muck and water. Ordinary earth lying below the ground-water, or rather ground-ice, is frozen throughout the summer, and the difficulties of working it, with the general sloppiness of the earthwork accomplished, are similar to those encountered in work in the very early spring in the northern part of the United States. It can be loosened, however, by blasting, and the difficulties in handling are merely those common to water-saturated earth. Earth above the ground-ice level thaws and dries readily, and its working is entirely normal.

The writer has traveled over a railroad near Nome to which the author possibly refers in the latter part of his paper. It was a very light surface road, and had the misfortune to be located, for the greater part of its length, on a frozen swamp. Light ditches had been dug through the moss and turf on each side, and these had gullied out so badly that much of the track had to be shifted to one side. The journey of about 60 miles occupied an entire day, although the passengers escaped the usual upset. The writer, however, does not think that any moral can be drawn from this railroad; a swamp does not afford the best location for a railroad in any country, and this road was merely fortunate enough to find its swamp frozen.

The prediction is ventured that good railroad construction in Alaska will follow standard practice closely, henceforth as well as heretofore. Swamps will be crossed on a fill, as usual. The principal departure from construction elsewhere will be in the matter of unit costs, and these, on account of the short working season and distance from supplies, will be several times greater than on any road in the United States.

Mr. Wheeler.

WALTER S. WHEELER, M. AM. SOC. C. E. (by letter).—Records at Skagway, Alaska, show that during the open season of 1907, the Guggenheims shipped into Dawson more than $5 000 000 worth of freight, a considerable portion of which was material for the Klondike pipe line of the Yukon Gold Company. The freight was landed, from ocean steamships, at the wharf in Skagway, and transhipped, over the White Pass and Yukon Narrow-Gauge Railroad, 111 miles to White Horse, and then by steamer down the Yukon to Dawson. The freight and passenger rates over this road are naturally quite high, as a considerable portion is built through exceedingly mountainous country, and the bulk of travel is confined to the summer, when navigation is open on the Yukon River. The track has a 3.9% grade for about 16 of the first 20 miles out of Skagway, to the famous White Pass, which is about 2 800 ft. above Skagway and mean tide. This portion of the road cost $2 000 000, or an average of $100 000 per

Mr. Wheeler.

mile, and it is said that the road more than paid for itself during the first two years of operation.

Mixed trains leaving Skagway are drawn by four engines distributed equally throughout the same. At a point 8 miles from Skagway, a rock thrown into the river from the track struck the water in 5 sec.

By the "Far North", the writer assumes that the author includes all of Alaska and the adjoining Yukon Territory, and also British Columbia lying directly east of Alaska.

Costs for material, labor, etc., vary considerably for different locations. At the time the Klondike pipe line was constructed, with common labor at $4 per day and board, the Alaska Road Commission was paying $2.50 per day and board on the trail out from Haines Mission, with a working season of 5 months in each location. Machinists were paid $5 per day in Skagway, with extra for overtime, and $1 per hour in Dawson, Yukon Territory, and the adjoining creeks. Cord-wood, the principal item of fuel, cost $5.50 per cord delivered in Skagway, and $15 per cord in Fairbanks, Alaska. Board and room, the best obtainable, could be had in Skagway for $40 per month, while the same class of board and room cost $75 per month in St. Michael, Alaska, and Dawson, Yukon Territory, and $125 in Fairbanks.

The cost of electric light in Skagway was 10 cents per kw-hr., and in Fairbanks, 51 cents. Business telephones cost $5 per month in Skagway, and $20 in Dawson and Fairbanks.

These prices were current about 1907 and 1908, and though some of them have changed from time to time, the writer has given them in order to show that they vary considerably in different locations of the "Far North", and that estimates for construction in the "Interior" are, as a rule, much higher than along the Coast.

A fair grade of coal could be purchased in Skagway for $13 per ton, but, in St. Michael, the writer has paid $25 for coal of the same grade. This coal was all shipped from the United States or from lower British Columbia, while billions of tons of the best bituminous and anthracite are waiting to be mined along the coast of Southern Alaska as soon as the Government sees fit to allow it to be opened up.

At Skagway the temperature seldom falls below zero, but it is not unusual to experience a temperature of —70° Fahr. in midwinter at Dawson, Fairbanks, and Nome, and sometimes —80° Fahr. When the temperature falls below about —40° Fahr., there is never any wind, and, the air being very dry at that temperature, the real danger to life is from exposure when it appears to be warmer than it really is.

In summer, ordinary vegetables are grown in abundance, and wild raspberries, blueberries, cranberries, and currants are plentiful.

An excellent breeding place for mosquitoes is formed below the niggerheads (bunches of coarse wild grass) on the tundra, and during

Mr. Wheeler.

May, June, and July, it is impossible to work on the tundras without the protection of nets and gloves; but August and September are comparatively free from these pests, which are not fever-bearing in the North.

It is impossible to make foundations to any great extent with the peat taken from the tundras, as it will dry out and burn up in the numerous tundra fires during the summer, as mentioned by the author in the case of the Nome Railroad.

In places near fresh and salt water, where suction dredges are available, good foundations can be made by running the discharge pipe so that the material will be deposited on the tundra, provided the water is not so rough as to prevent the dredge from operating. The writer had charge of dredging the St. Michael Canal during the summers of 1908 and 1909, and the material discharged from the pipe line would have made excellent foundations on the tundra, if they had been desired. When carrying on construction near the water, it is well to quarter the men on houseboats anchored well out from shore, so that, while not on duty, they can get some respite from mosquitoes. In this way one can obtain a more efficient working crew.

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PAPERS AND DISCUSSIONS

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CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION.

Discussion.*

BY MESSRS. ALLEN HOAR AND W. C. HAMMATT.†

Mr. Hoar.

ALLEN HOAR, JUN. AM. SOC. C. E. (by letter).—In Mr. Hammatt's very interesting paper, he has brought out very clearly that in California there is an abundant supply of first-class road material of all kinds, well distributed over the entire State, and that, therefore, poor roads there are inexcusable. The author, however, makes several statements, which, though true, in a sense, must be misleading to those who are not thoroughly familiar with all the conditions. The writer is pleased to note that Mr. Hammatt has spoken so highly of water-bound macadam. This type of pavement when well-drained, laid on a properly prepared sub-grade, and carefully constructed, has still a great deal of usefulness.

Under the heading, "Oiled Macadam," the author says, "A very small proportion of the total area of oiled macadam pavement constructed has been successful." He also says, "even under the best conditions, this pavement has little justification." Now, as a matter of fact, as there has been some success, there is certainly some justification for its use. In the introduction to the paper Mr. Hammatt states that it has been only in the last 5 or 6 years that any real road construction has been undertaken in California, but that a great deal of graveling and oiling had been done as a temporary improvement. Such is the case, the first practice of oiling being merely to afford

*This discussion (of the paper by W. C. Hammatt, M. Am. Soc. C. E., published in April, 1914, *Proceedings*, and presented at the meeting of May 20th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

†Author's closure.

Mr. Hoar. a temporary relief from the dust nuisance and to protect earth and gravel roads from ravages by heavy rain storms. Observation of the results obtained by this treatment led to the adoption of this material in road construction, and finally to the present perfected type of bituminous macadam. In making so broad a statement, that only a very small proportion of the oiled macadam roads has been successful, the author has presumably taken stock of all those old oiled dirt roads, only a few of which can be considered at all as oil macadam.

It is true that at first there were no fixed specifications for work of this class. It was then in the experimental stage, and each engineer was doing his own experimenting at the expense of city, county, or State. In Los Angeles County alone probably $1 500 000 were practically wasted during this experimental stage, simply because the engineer then in charge refused to profit by the experience and advice of others, but would learn for himself. Other cases of the same kind have occurred, and these unfortunate conditions have done much to blacklist pavements of this type. At present engineers engaged in work of this class have adopted similar specifications and are obtaining general success.

The author states that the successful examples of pavements of this type have been either in localities where climatic conditions are such as to evaporate the more volatile parts of the oil, or where the application has been made in such a manner as to obtain the same results. Now, if success is obtained by a certain method in one place, would it not be good practice to adopt such a method in other localities? As a matter of fact, an asphaltic oil which will meet the requirements of the usual specifications as now adopted has practically no lubricating properties and very little volatile matter. The foundation stones are laid and rolled until a thorough mechanical locking of the particles takes place and it is as firm and solid as a water-bound macadam pavement; then the oil, at a high temperature, is sprayed on under pressure in such a manner that it is in a finely divided state and penetrates the interstices of the stone, acting primarily as a binder. Constructed in this manner, there can be no rolling or movement of the stones, as is pointed out by Mr. Hammatt. The wearing surface of finer aggregate is then laid on this foundation.

The patented form of Petrolithic pavement is by no means an oil macadam pavement. It is merely an earth road with sand and oil incorporated to a depth of 6 or 8 in., forming a thick rubbery mat, which is pushed and pulled by the traffic stresses until it is resolved into a series of humps and hollows. In a hot climate it remains soft and spongy, and soon becomes so wavy and covered with ruts that it is almost impassable. The Petrolithic Company's tamping roller, however, has come into very good use in compacting sub-grades and earth roads. The prongs, or sheep's feet, of this roller reach through

to the bottom of the sub-grade, and, starting there, compact it uniformly all the way to the surface, making a solid crust or arch to carry the weight of the finished pavement and its transmitted stresses. Mr. Hoar.

Probably the strongest argument against bituminous or oil-macadam pavements is the difficulty of obtaining good inspection, which must be had to secure satisfactory results. On the contrary, an argument put forth by the advocates of concrete roads is that, because of this lack of trustworthy inspection, concrete is the only type from which satisfactory and certain results can be obtained. In practice, however, this has proven to be far from the truth. The argument is that, no matter how little care has been taken in the preparation of the sub-grade, or how poorly the concrete has been mixed, it will become hard and form a good solid foundation, and will carry the traffic safely over the bad places in the sub-grade. Experience, however, has shown that, to secure satisfactory results with concrete roads, just as much attention must be given to the inspection as for any other type; for lack of the proper quantity of cement or poor and insufficient mixing is soon indicated by rapid disintegration.

The type of wearing surface which has met with the greatest success on roads having a concrete base consists of a $\frac{3}{8}$-in. coat of oil and crushed stone screenings ranging in size from $\frac{3}{4}$ to $\frac{1}{4}$ in. The oil is thus almost uniformly exposed to the action of the atmosphere, which causes it to "set", thus cementing the particles of the aggregate well together and to the base. As the coat is thin and uniform, it is easily applied, cheaply maintained, and permits of equal compression throughout the mass.

W. C. HAMMATT, M. AM. SOC. C. E. (by letter).—As was expected, the writer's condemnation of oiled macadam pavements brought out some contradiction. Although this type of pavement has lost many of its advocates, there still remain many who believe in its value as a pavement. The writer, however, maintains that only a small proportion of the total mileage of oiled macadam pavement which has been laid, has been successful, and he does not include the oiled dirt and gravel roads under this heading. He knows of no road in the central part of the State, which has been under varied traffic for two years, which has remained in good condition without an expenditure of at least 8 cents per sq. yd. for repairs. There are some, where traffic is restricted to pleasure vehicles, which have remained in good condition, and these are cited as examples by advocates of this class of pavement. Mr. Hammatt.

The value of oil as a road material lies in the formation of a surface skin by the volatilization of the lighter constituents. This skin is very durable, and as long as it remains intact, it protects the general body of the pavement. In the southern part of the State, where Mr. Hoar's observations have largely been made, the sandy

Mr. Hammatt.

nature of the soil and the hot, dry climate are such as to increase the thickness of the skin. Nevertheless, in examining the roads between Los Angeles and Pasadena, the writer has noted that those carrying the automobile traffic are the only ones which hold their shape.

To sum up, the justification for the adoption of a certain type of pavement can only be from its structural excellence or its economic value. One type will not be justifiable if another type will make a better appearance and keep in better condition at a smaller cost. The writer's experience has been that a protected water-bound macadam pavement will do this. This type may be made to fit all conditions by varying the thickness of the base and of the protective covering. A macadam pavement, 6 in. thick, with a skin protection of asphalt and screenings, will cost about 70 cents per sq. yd., and will require an annual expenditure of about 9 cents per sq. yd. for upkeep. Increasing the protective covering to 2 in. in thickness of asphaltic macadam increases the cost to about $1.10 per sq. yd., and the upkeep will be nil for from 10 to 15 years. Compare these costs, as well as the appearance and usefulness of the pavement, with oiled macadam under the best conditions, and the writer believes that there will be no justification for the latter.

A skin of oil and screenings has been applied to some of the concrete bases of the State highway for their protection. This has been quite successful, as pointed out by Mr. Hoar, for the reason that the oil does not penetrate the concrete, and the concrete, by absorbing heat, promptly carbonizes the oil and hardens the skin. Moreover, for so thin a coating, it is probable that oil is better than asphalt, as the coating will be more resilient and cause less impact on the concrete.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EMIL GERBER, M. Am. Soc. C. E.*

DIED APRIL 16TH, 1914.

Emil Gerber was born in Reichenbach, Saxony, on January 31st, 1858, and died in Pittsburgh, Pa., on April 16th, 1914. His father, C. F. Gerber, was born in 1819 and died in 1899. He was a designer and manufacturer of textile fabrics, and introduced the use of power looms in his native town. His mother's maiden name was Christliebe Klotz, daughter of Carl Klotz. His ancestors for several hundred years were residents and prominent citizens of Reichenbach, Saxony. His father came to the United States in 1862 and settled in Webster, Mass., and in 1867 his mother came to Webster, bringing Emil and his two brothers, Herman and Carl.

He was educated in the common schools of Webster, Mass., and the Stevens High School at Claremont, N. H. In the fall of 1873 he entered the Worcester Polytechnic Institute, at Worcester, Mass., and was graduated as a civil engineer in the spring of 1876, being at that time only a little more than eighteen years of age. A classmate states that he was very much liked by all the students and professors. He was studious, a man of high character, and at that time gave promise to do good work. The President of the Institute writes: "Few students have done better in the Institute."

Mr. Gerber taught school for one year at Southbridge, Mass., and was employed as a bookkeeper in a corporation store connected with the woolen mills in Webster, Mass., until May 1st, 1879, when he was engaged as a Transitman on the Fremont, Elkhorn, and Missouri Valley Railroad. In May, 1880, he became Locating Engineer with the same railroad, and in 1881 was promoted to the position of Assistant to the Chief Engineer, Captain James Edward Ainsworth. On August 1st, 1881, he was appointed Assistant Engineer on the Blair Bridge over the Missouri River, near Blair, Nebr., built by the Sioux City and Pacific Railroad which later became a part of the Chicago and Northwestern Railway. The late George S. Morison, Past-President, Am. Soc. C. E., was Chief Engineer of this bridge. Toward the completion of this work, on November 1st, 1883, Mr. Gerber was made Resident Engineer.

In October, 1884, he returned to his former position with the Fremont, Elkhorn, and Missouri Valley Railroad, but still retained charge of the Blair Bridge and its extensive protection works.

* Memoir prepared by Otis E. Hovey and August Ziesing, Members, Am. Soc. C. E.

In November, 1885, he was appointed Assistant Chief Engineer of the same railroad, which position he held until he resigned on June 15th, 1887, to become Resident Engineer of the Sioux City Bridge over the Missouri River, of which Mr. Morison was also Chief Engineer. This bridge was completed at the end of 1888, and Mr. Gerber was then appointed Resident Engineer of the bridge over the St. Johns River at Jacksonville, Fla., which was being built by the Jacksonville, St. Augustine, and Halifax River Railroad, with Mr. Morison as Chief Engineer. A severe illness prevented Mr. Gerber from completing this bridge, and in May, 1889, Mr. Morison placed him in charge of his Chicago office, where he remained until the fall of 1897.

During this period, he was connected with all Mr. Morison's important works, among which may be mentioned the Cairo, Memphis, Burlington, Winona, Bellefontaine, Alton, and Leavenworth Bridges; the Chicago, Burlington, and Quincy entrance into St. Louis, and many less important works.

In 1897, Mr. Gerber resigned his position with Mr. Morison to become Chief Engineer of the Lassig Bridge and Iron Works, in Chicago, Ill. He held this position until the spring of 1900. When the American Bridge Company was formed, he was made Manager of the Lassig Plant. In July, 1901, he was promoted, becoming Assistant to the President of the American Bridge Company, in Philadelphia, Pa. Early in 1904, he went to Pittsburgh, in the same position, and in the fall of 1905 was made Operating Manager of the Pittsburgh Division in addition to his position as Assistant to the President. In 1911, he was relieved as Operating Manager and made General Manager of Erection, but still held the title of Assistant to the President. He held this position during the remainder of his life.

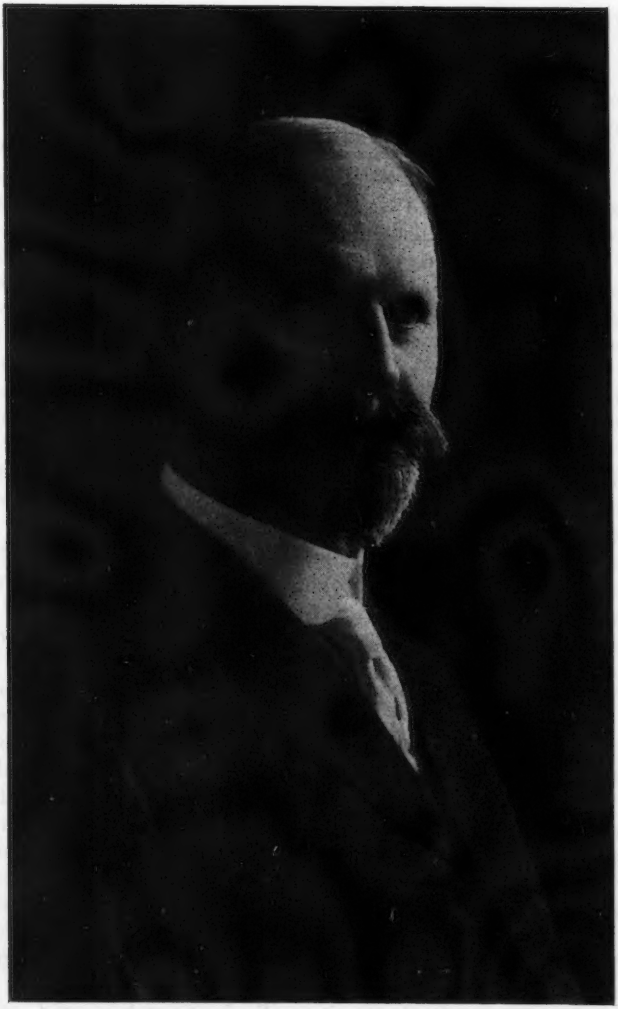
He was married on January 3d, 1882, at West Roxbury, Mass., to Caroline Herthel, daughter of F. J. Herthel, Sr., and is survived by his widow, a daughter, Mrs. Laura E. Olson, of Chicago, Ill., and a son, Emil Gerber, Jr., of Pittsburgh, Pa.

As is often the case with engineers fully and continuously engaged on important work, Mr. Gerber was not a frequent contributor to engineering literature, but whatever he wrote is strongly marked by his vigorous personality.

He was the author of a paper entitled "Painting of Iron Structures Exposed to Weather",* and also of one entitled "Some Commercial Features of Structural Engineering".† He had based a few lectures to young engineers on the latter paper, and his last brief illness and death prevented him from lecturing on the same subject before the American Bridge Engineering Club of New York. This paper is one which should be read by all engineers.

* *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 185.

† *Proceedings*, Eng. Soc. of Western Pennsylvania, Vol. XXIII, p. 125.



E. Gerber

It frequently happens that an engineer employed by individuals or corporations does not become so well and favorably known as he would have been had he been in practice under his own name. It is also common for an engineer associated with corporations to specialize along some narrow line of engineering work. Mr. Gerber was an exception in these respects. His early experience covered a very wide range of work, including railroads, difficult foundations, construction of bridges and other structures pertaining to railroads, and, at the time he took up his work as an engineer associated with a corporation, his experience was wide and his judgment well trained.

Those closely associated with him were impressed by the thorough manner in which he carried out any work on which he was engaged, his genial and helpful disposition, and his hearty and liberal endeavors to assist those around him in every way. He was a man who would ungrudgingly draw from his experience anything that would help his associates to grasp an engineering or business situation with which he was familiar.

He never did anything superficially. Any problem that was before him received his complete and earnest attention. Whenever he undertook an investigation or made a report, one could depend on his having gone into the matter thoroughly and having expressed his best opinion; and any facts contributed to the subject could be depended upon.

Mr. Gerber had a strong mentality and firm opinions on any subject to which he had given thought, but at the same time was always willing to discuss exhaustively the opinions of others, and when they were backed by facts and good reasoning was ready to modify his previous views in an eminently fair-minded manner.

A notable trait of his personality was his unusual ability in the establishment of cordial relations with his associates, and his interest in all that contributed to their welfare.

On account of his connection with a multitude of engineering works to which he contributed in more or less degree, and in which his strong common sense and clear thinking determined important decisions concerning designs and methods of construction, it is unfortunate that the Profession cannot point definitely to this or that work as purely his own. His monument is a more personal one. His strong, honest, thorough, and genial personality has impressed itself on all who knew him, and most of all on those who knew him intimately. Such characters are an inspiration to all members of the Profession.

Mr. Gerber was elected a Member of the American Society of Civil Engineers, on February 1st, 1888, and served as a Director from January, 1912, until the time of his death. He was also a member of the American Railway Engineering Association, the Western Society of Engineers (Past Treasurer), the Engineers' Society of Western Pennsylvania, the Chicago Engineers Club, and the Duquesne Club and the Junta Club, both of Pittsburgh.

HENRY FRANCIS LABELLE, M. Am. Soc. C. E.*

DIED DECEMBER 12TH, 1913.

Henry Francis Labelle was born in Montreal, Que., Canada, on January 31st, 1860. His father, Regis Labelle, was a lumber merchant, and was a native of Jirome, in the Province of Quebec. His mother, Lucille Marceau, was born at Lacolle, in the same Province.

After his graduation from St. Mary's College, Montreal, he entered the Montreal Polytechnic School, from which, in 1882, he received the degree of Civil Engineer.

Mr. Labelle's first employment was as Rodman and Draftsman for the Chicago, Burlington and Quincy Railroad. In August, 1883, he returned to Canada, and, after about a year in the Dominion Lands Office, entered the office of the Chief Architect, Department of Public Works, Canada, where he was employed until 1890, during the last two years as Assistant Superintendent of Public Buildings. From 1890 to 1895, he was employed as Assistant Engineer or Engineer in Charge of the design and construction of numerous municipal water supply systems in New York, Pennsylvania, and Maryland.

By disposition and training Mr. Labelle was peculiarly fitted for the varied and interesting problems of hydraulic and hydro-electric engineering design, and after once entering this branch of the Profession, he did not leave it.

He was for several years in the employ of the East Jersey Water Company, the latter part of the time as First Assistant Engineer in charge of the Montclair office and of construction work amounting to approximately $1 000 000. In 1895, and again in 1900, he was employed by the Susquehanna Power and Paper Company, on surveys and designs for a 35 000-h.p. development on the lower Susquehanna River, and in 1902 by other parties on investigations and studies of power possibilities on the same river at Turkey Hill.

In 1901, he was appointed Designing Engineer on a new water supply for Santiago, Cuba, for the Military Government. In the latter part of 1902, he returned to Cuba where, for some time, he was engaged by Havana interests as Engineer to report on or design water supply and electric power and light systems.

In 1904 Mr. Labelle accepted employment under the Philippine Government, first as Engineer in Charge of all irrigation work in the Islands, under the Director of Public Works, and later as Principal Assistant Engineer on the construction of the new Manila Water-Works. While in charge of the irrigation works, he prepared a system of irrigation laws for the Islands.

In 1907, on account of threatened lung trouble, he was advised by his physicians to leave the Philippines. He returned to the United

* Memoir prepared by John T. Whistler, M. Am. Soc. C. E.

States, and located in Silver City, N. Mex., where he remained until 1911, when heart trouble compelled him to change from the high altitude to a lower one, and he moved to Albuquerque, N. Mex., where he remained until his death.

It was typical of his character that he wrote of his heart trouble in 1911, "I now have the choice of two ways to die. I can either remain in Silver City and die of heart trouble, or I can go to Albuquerque and die of lung trouble."

After several months of correspondence and inquiry as to his experience and qualifications, he was notified, on November 5th, 1911, of his employment by the Argentine Government to act as Adviser in the development of an Irrigation Bureau. It was just at this time that his complication of heart trouble had developed, and, much to his regret, he was compelled to decline the office.

In 1908, he was married to Miss Emily Rose Woodford, of London, England, who for several years had been a resident of Philadelphia, Pa., and who survives him.

Mr. Labelle was the author of several discussions published in the *Transactions,* and had also contributed to various current technical journals. Though of foreign birth, he took out naturalization papers and became a citizen of the United States in January, 1904.

Altogether, Mr. Labelle was one of the sweetest and most noble characters with whom it was one's privilege to associate. He never complained, rarely spoke ill of any one, and then only with that calm judgment which guided his whole career. He never was known to do any one the slightest injustice. Simply to have known him was a privilege, to have known him intimately was an inspiration.

Mr. Labelle was elected a Member of the American Society of Civil Engineers on April 6th, 1898.

FRANK PARSONS LANT, M. Am. Soc. C. E.*

DIED JUNE 30TH, 1914.

Frank Parsons Lant was born at Nassau, N. Y., on May 24th, 1858, and was of Colonial Dutch descent.

His business career, which included many important positions, was begun on September 26th, 1881, when he was employed as Axeman, and, subsequently, as Rodman and Assistant Engineer on the location and construction of the West Shore Railway, holding the latter position until March 1st, 1884.

From June 1st, 1884, to December 31st, 1886, Mr. Lant was City Engineer and Surveyor of Hudson, N. Y., designing and constructing

* Extract from memoir prepared by Gerald E. Hart, Esq., of New York City and Jacksonville, Fla., supplemented by information on file at the Society House.

sewers, highways, etc. He was also engaged in the private practice of surveying and engineering during that time.

During 1887, and until March 1st, 1888, he was employed on the location of a branch line for the Erie Railway, in Pennsylvania, and also made the surveys and had charge of the construction of the foundations for the Myrtle Avenue Elevated Railroad, in Brooklyn, N. Y. During this period he made the preliminary and location surveys for the New York and Massachusetts Railroad, from Ancram Lead Mines, N. Y., to near Westfield, Mass.

From March to November, 1888, he served as Division Engineer on the preliminary and location work for a line over the Blue Ridge Mountains, north of Marion, N. C., for the Charleston, Cincinnati and Chicago Railway, and from November, 1888, to March, 1889, he was engaged on a revision of the location of the New York and Massachusetts Railroad from Ancram Lead Mines, N. Y., to Chicopee Falls, Mass. In April, 1889, he was placed in charge of the reballasting of 60 or 70 miles of roadbed for the Rome, Watertown and Ogdensburg Railroad.

In November, 1889, Mr. Lant went to Jamaica, West Indies, where he remained until July, 1891, making location surveys for the West India Improvement Company.

From August, 1891, to February, 1896, he was engaged in the private practice of engineering and surveying, during which time he made the plans for the disposal of ashes in the filling in around Riker's Island, a survey for a railway proposition on Long Island, etc., etc.

In connection with the public works of New York City, Mr. Lant served the City Government, as follows: From February, 1896, to October, 1897, he was Engineer Inspector in the Department of Public Works; from October, 1897, to July, 1898, Engineer of Grades (Commissioners' Engineer) on the electrification of the Second Avenue Railroad; from August, 1898, to February, 1902, Transitman and Computer in the Topographical Bureau of the Board of Public Improvements; and from February, 1902, to March, 1907, Transitman and Computer in the office of the Engineer of Street Openings.

In March, 1907, he was appointed Manager (which title also included that of Chief Engineer and Surveyor) of the Lawyers' Engineering and Surveying Company, of New York City, which position he held at the time of his sudden death.

Mr. Lant was married on November 11th, 1891, to Stella, daughter of the late Jacob L. Seixas, who, with his father and mother, survives him.

He was an ardent student in his chosen profession, to which he did honor. His capabilities were such that his services and advice were frequently sought in cases of special importance. He possessed

in a high degree the power to command loyal service from his subordinates, as well as the confidence and respect of all who knew him. His genial manner and kindly nature will long be remembered by many who, in his death, experience a sense of personal loss.

Mr. Lant was elected a Junior of the American Society of Civil Engineers on October 3d, 1888, and a Member on February 1st, 1910.

ULYSSES STANISLAUS LUTZ, M. Am. Soc. C. E.*

DIED DECEMBER 8TH, 1913.

Ulysses Stanislaus Lutz, the son of Ignatius and Rose Jacwuin Lutz, was born in Philadelphia, Pa., on May 25th, 1853. He received his earlier education at Roth's Military Academy, and was graduated in 1873 at the head of his class from the Philadelphia Polytechnic Institute, with the degree of Civil Engineer.

In 1873 he was engaged as Rodman on the South West Pennsylvania and the Lewisburg, Centre, and Spruce Creek Railroads. In 1874 he served as Leveler on the Tyrone and Clearfield and New River Railroads; in 1875 as Rodman on the Pennsylvania Railroad; in 1876, as Transitman on retracing the old Portage Railroad; from 1877 to 1879, as Assistant Supervisor, Maintenance of Way, on the Pittsburgh and Philadelphia Division, of the Pennsylvania Railroad; in 1880, as Supervisor, Maintenance of Way, on the Shenandoah Valley Railroad; from 1881 to 1885, as Assistant Engineer on Surveys for the South Penn Railroad, for two years, and for two years as Resident Engineer in charge of Subdivision B, Division 5, of the South Penn Railroad, in Somerset County; in 1886, as Principal Assistant Engineer on the location and construction of the Bloomsburg and Sullivan Railroad; in 1887, as Principal Assistant Engineer on the location of the Baltimore and Drum Point Railroad; in 1888 as Construction Engineer of the 25th Ward Gas Works, in Philadelphia, Pa.; in 1889, as Division Engineer on the location and construction of the Cumberland Valley Extension of the Louisville and Nashville Railroad.

For a time Mr. Lutz followed private practice in Virginia and North Carolina, and, in 1892, became Constructing Engineer for the Pennsylvania Iron Works. In 1893 and 1894, he was Chief Engineer on the location and construction of the South Jersey Railroad, and thereafter was engaged on the construction of electric railroads in and around Baltimore, Md., having been, in 1895 and 1896, Manager of Construction for the Baltimore and Catonsville Railroad.

* Memoir prepared by Charles W. Staniford and S. L. F. Deyo, Members, Am. Soc. C. E., and John H. Frazee, Assoc. M. Am. Soc. C. E.

In 1897, he entered the service of the City of New York in the Department of Public Works, being first engaged on the Park Avenue Change of Grade Improvement from 56th to 96th Streets in Manhattan.

Mr. Lutz left the City's service to go with his former Chief, the late William F. Shunk, as Chief Assistant Engineer on the location and construction of the Quito and Guayaquil Railroad, in Venezuela, and again to go with the Iowa and Minnesota Railroad, as Assistant Engineer on Construction.

In 1905, he entered the Department of Finance of the City of New York, as an Assistant Engineer, and held that position until his death, which occurred on December 8th, 1913, after an illness of three weeks.

Mr. Lutz's practical knowledge acquired through a long and varied engineering experience was of great service to the City. He was an indefatigable and conscientious worker, and those with whom he was associated were impressed with his sense of loyalty to the City and to his friends.

Mr. Lutz was elected a Member of the American Society of Civil Engineers on March 5th, 1912. He was also a member of the Masonic Fraternity.

LEON LINCOLN GAY, Assoc. M. Am. Soc. C. E.*

DIED MAY 6TH, 1914.

Leon Lincoln Gay, the only son of Carlos E. and Calista Gay, was born at Barton Landing, Vt., on May 30th, 1879. He obtained his education at the public schools, and was a graduate of St. Johnsbury Academy. He then entered the Sheffield Scientific School, of Yale University, from which he was graduated in 1901, with the degree of Ph.B. While at Yale, he was a member of the Track Team and made some good records as a long-distance runner.

In the fall of 1901, Mr. Gay went to Idaho to become Assistant Engineer in charge of the construction of a small power plant at Horseshoe Bend, under A. J. Wiley, M. Am. Soc. C. E., Consulting Engineer. On the completion of this plant, he entered the United States Reclamation Service as Engineering Aid, and was engaged on the preliminary work for the Minidoka Project, in Idaho, during the greater part of 1903 and 1904. During the construction of the Minidoka Dam, on Snake River, he was Assistant Engineer on that work. On its completion, he was assigned to take charge of the building of the original Jackson Lake Dam, near Yellowstone Park, in Wyoming, and made an enviable record in completing this temporary

*Memoir prepared by C. W. Joslyn, Esq., and F. C. Horn, M. Am. Soc. C. E.

dam at a remarkably low cost. Later, he was Principal Assistant Engineer in charge of the construction of the Boise River Dam, of the Reclamation Service, near Boise, Idaho.

In 1908, Mr. Gay resigned from the Reclamation Service to accept a position as Engineer for a large irrigation enterprise in the Sacramento Valley, California. The following year he was attacked with tuberculosis, and the last five years of his life was a continuous fight with this dread disease to which he finally succumbed while undergoing an operation in Montreal, Que., Canada.

Mr. Gay was known by the large number of engineers of his acquaintance in the West to be a man of high moral character and absolute integrity. He was considered one of the most capable young engineers in the line of irrigation in the United States, and previous to his unfortunate illness he had executed some important and successful work. His misfortune and death are peculiarly sad. He was a typical New Englander, and among his near friends, his humor and originality gained for him the affectionate nickname of "the Yank".

He is survived by his father and stepmother, of Orleans, Vt., as well as by his two sisters, Mrs. Wallace C. Gilpin, of Barton, Vt., and Mrs. Fred E. Parker, of Falls Church, Va.

Mr. Gay was elected a Junior of the American Society of Civil Engineers on December 5th, 1905, and an Associate Member on October 2d, 1907.

PHILIP MORRIS PRITCHARD, Assoc. M. Am. Soc. C. E.*

DIED JULY 8TH, 1914.

Philip Morris Pritchard was born on April 29th, 1872, at Rock Ferry, near Birkenhead, England. He was educated in London, where his father was in business as a tea importer and merchant. At an early age the boy showed a strong liking for Engineering, and after his graduation from Kings College, London, in 1891, he was sent to Newcastle-on-Tyne to serve an apprenticeship of four years with the firm of R. and W. Hawthorn, Leslie and Company, Limited, Locomotive and Marine Engineers and Shipbuilders, where he received a thorough practical workshop training. During three years of his apprenticeship, he attended the Engineering Course at Durham College of Science, at Newcastle, and, on his graduation, was awarded an Associateship in Science.

In June, 1895, Mr. Pritchard was engaged as Draftsman and Assistant Engineer at the Tennant-on-Tyne Works of the United

* Memoir prepared by the Secretary from information supplied by the United Alkali Company, Limited, Widnes, England, supplemented by information on file at the Society House.

Alkali Company, Limited. In January, 1896, the Company organized its Engineering Department at Widnes, England, and he was brought from the Tennant Works to occupy an important and responsible position on the engineering staff at the latter place. This promotion eventually led to his appointment as Chief Engineer of the Company, a position which he held for about 12 years and at the time of his death on July 8th, 1914.

While in charge of the Engineering Department of the United Alkali Company, Limited, Mr. Pritchard was engaged in the design and erection of much new plant in the English and American works of the Company and did valuable work in its Spanish mines.

All through his career, Mr. Pritchard showed brilliant ability as an Engineer, and his death is a great loss to the Company as he was greatly esteemed both by the Directors and his colleagues.

He was a Member of the Institute of Mechanical Engineers and of the Society of Chemical Industry. He was also an Associate Member of the Institution of Civil Engineers.

Mr. Pritchard was elected an Associate Member of the American Society of Civil Engineers, on June 6th, 1900.

PAPERS IN THIS NUMBER

"WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT." H. T. CORY. (To be presented Nov. 4th, 1914.)

"THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA." E. L. SAYERS and A. C. POLK. (To be presented Nov. 4th, 1914.)

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FUNDAMENTAL PRINCIPLES OF PUBLIC UTILITY VALUATION.

By John W. Alvord, M. Am. Soc. C. E.

To be Presented November 18th, 1914.

Synopsis.

This paper is undertaken as a consequence of the voluminous discussion of the writer's prior paper: "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return,"* presented December 17th, 1913. That discussion, though it showed the greatly increased interest which valuation matters are now receiving, also seemed to show that many engineers lack a clear perception of the fundamental principles involved in valuation work, presumably because such principles are to be found only in the law and the interpretation of the law by the Courts.

This paper shows that the science of valuation calls for a knowledge in three professions or callings: the law, engineering, and economics, and that, fundamentally, it is the law, and its interpretation by the higher Courts, which controls.

The fundamental definitions of value and property are given, and the constitutional conception of what it is that is to be valued

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

is shown, with citations from leading cases passed on by the Supreme Court.

The point is made that the law determines that it is a "property" rather than an actual past investment that is to be valued, and that the value must be found as of to-day.

The fact that we are to value a property as of to-day indicates why the reproduction method obtains its importance, but it is made clear that reproduction is only a cost method, that cost is not value, and that the Courts have indicated that all lines of material evidence should be examined.

An outline is given of the various lines of evidence that should be summed up, including evidence that cannot be expressed well in figures.

Reproduction is thus examined only as a line of cost evidence, and its conceptual requirements are discussed, illustrated, and summarized. Past cost is also discussed as a line of cost evidence, and its deficiencies in throwing light on value now are pointed out; misuse of cost evidence methods is explained, other lines of evidence not readily reduced to cost figures are described, and, finally, the importance of reasoning from cost lines of evidence—as well as other evidence—to value is emphasized and illustrated.

To summarize, it has been attempted by this paper to show:

First.—That it is quite idle for engineers, appraisers, utility commissions, and others to argue about the detailed methods of valuation unless they have a fundamental and uniform conception of what "value" and "property" are, in the first instance, and what the reasoning of the Courts has been on these subjects.

Second.—That, fundamentally, the art of valuation must rest in the law and the interpretation of the law by the Courts, and especially the Supreme Court of the United States, this being the final authority in valuation matters.

Third.—That the Supreme Court has repeatedly ruled that it is the value of the property used for the public purpose that must be considered, and it is the value of that property now.

Fourth.—That most of the utility commissions and many estimators who have been following their rulings are at variance with the opinions and decisions of the Courts, in that they are insisting on retracing an investment rather than valuing a "property."

Fifth.—That the reproduction method is in most cases a most useful line of inquiry in arriving at a conclusion as to what the value of a property may be at the present time, but that it is not the only line of evidence to be considered.

Sixth.—That reproduction is a cost method and does not of itself determine value. It is only one of the means; not an end.

That the reproduction must be complete in itself, including going costs, before it can be properly used to reason from.

Seventh.—That when all lines of evidence are properly before us, we must reason as to what bearing on value each one of them may have.

A list of important legal decisions is appended to the paper.

Introductory.

The problem of valuing the property of public utilities is a difficult one, largely on account of the fact that there is required a knowledge of certain fundamental principles of law, of engineering, and of economics or finance. No one of these professions has within itself all the training and information which must be brought to bear on the problem, in order to make an equitable valuation. Experience and information from all of them must be utilized to ensure sound conclusions.

The Courts are somewhat prone to criticize expert witnesses because they do not agree, forgetting often that the fundamental data are indeterminate; financiers and economists are impatient because judges and lawyers do not appreciate the natural laws of business and the reasoning of sound economics; and, finally the engineering specialist becomes irritated with the Courts because his views on such subjects as "property rights," "franchises," "past cost," and "present value" are not deferentially treated, little realizing that he is here speculating entirely within the domain of the legal profession, and is often guilty of attempting to make law, where much more competent and well-trained minds have long been at work in the endeavor to produce sound reasoning. The engineer is especially impatient because the economist sees no practical difficulties or uncertainties in construction, and both the economistic and legal type of mind are too prone to regard the engineer as primarily a mathematician, who

should produce exact results even from inexact data, little realizing that engineering work is often more judicial than mathematical in its ordinary application and practice.

It must be apparent that it is quite useless to endeavor to discuss intelligently the details of valuation work unless somewhere the underlying principles, both as regards engineering and the law and economics, are co-ordinated, and their relative value to the problem in hand is fully appreciated. It is the purpose of this paper to attempt this somewhat ambitious task.

Definition of Terms as Used in this Paper.

In the present state of confusion as to valuation nomenclature, it is desirable to define the meaning of the more important terms used in this paper, as follows:

Property.—That which is owned; that which belongs exclusively to an individual; that to which a person has a legal title (whether in his possession or not); the exclusive right of possession, including all the rights which accompany ownership and is its incident.

Value.—The sum of the desirable qualities which render a thing useful and sought, measured in money; a measure of the relation of two services growing out of the adjustment of mutual needs.

Market Value.—The adjustment of two services, *i. e.,* as between a willing seller and a willing buyer under open conditions of competition.

Cost.—The actual outlay of money, or its equivalent, for a property, structure, or machine.

Past Cost.—The amount of money, or its equivalent, actually expended in the past in creating and building up a property.

Investment.—The amount of capital, or its equivalent, actually expended for a property in the past.

Reproduction Cost.—(Used in this paper generally as a short term for reproduction less depreciation.) An estimate of the cost of recreating a property at the present time under conditions that are humanly possible and practical.

Depreciation.—The lessened value of any property, structure, or machine, due either to its wear, loss of usefulness, growing lack of adaptation, or approaching abandonment.

Appreciation.—The increase in worth of a property, structure, or machine, due to its increasing use, strategic location, the increasing need for its service, or other like influences.

Amortization.—The repayment of an original investment or debt by means of sinking funds, or other moneys set aside from time to time in expectancy of obsolescence.

Public Utility.—A business supplying a public need, and based on a public grant.

Monopoly.—A business having exclusive power of dealing in a service, and thus conducted without competition.

Franchise.—A grant by the public of the necessary rights to do a specific business.

Other definitions might be included, pertaining more to the details of appraisal work, but as it is the intention here to limit the discussion as far as possible to fundamentals, it is believed that the foregoing will suffice.

Law—The Foundation.

The fundamental principles of the science of valuation must rest primarily on the law, and to an adequate knowledge of the law must be added a knowledge of the interpretation of the law by the Courts in their decisions and opinions. Courts are not free to reach conclusions on broad general principles; their function is, not to say what the law ought to be, but simply to interpret it as it is. They, as well as economists and engineers, must accept fundamental law as it stands; their duty is to tell us what this fundamental law means.

The function of the lawyer, therefore, in utility valuations, is to call the attention of engineers, economists, and appraisers to the law and to the interpretation of the law by the Courts, and to assist in its application to the facts, and this is a fundamental and important matter. What is to be determined, and the material factors to be taken into account in its determination, are questions of law, exclusively. It is in this connection that the utility commissions, the economist, and the engineer usually make their most serious mistakes. Disregarding the legal basis on which all valuation work must rest, their attempts to decide difficult and important legal questions on the strength of their own general judgment, knowledge, or practice, alone, without any inquiry into the law governing such questions,

naturally adds seriously to the confusion and difficulties surrounding an already complex problem.

The final authority for the interpretation of the law, which is the foundation on which the work of public utility valuation in this country must be built, is the Supreme Court of the United States. Its decisions, until modified, must be regarded as authoritative and final. Next in importance, in interpreting the law, come the decisions and opinions of the higher State Courts, which have been studying and applying the law to questions of this kind for the last fifteen or twenty years, producing an historically connected body of reasoning which no serious student of valuation work can afford to remain in ignorance of or neglect. The utility commissions are all subject to review by the higher Courts, and are, therefore, in effect specialized lower Courts or legalized appraisal boards with enlarged and extended powers.

The interpretation of the law by the Courts has in it little of permanent value, unless it expounds the fundamental law along the lines of sound reasoning and justice. With great responsibilities resting on them, Courts naturally proceed cautiously, rarely stepping outside of the necessities of the particular case in which they have to produce a finding, and yet progressing step by step, so that, as new applications of old principles are needed, there is slowly built up a rational line of reasoning which is of the greatest fundamental value. It must be admitted that Courts ofttimes get confused and bewildered when unexpectedly called on to determine purely economic or technical questions. It is not unjust to say that in some cases judges are unfitted by prior training and so handicapped by lack of time as to prevent them from mastering the economic or technical problem before them. Under such circumstances, they sometimes make technical mistakes in their decisions, which are unfortunate because they delay progress. When a Court is bewildered by contradictory expert testimony, it occasionally reveals its embarrassment by criticizing the experts. Courts should realize that truth often lies in a twilight zone, and that this twilight zone may be much narrowed down by the services of good, specialized, and competent witnesses in their calling, and Courts ought to be so skilled in human nature as to be able to disassociate the able and mature from the immature and pretentious witness on specialized subjects.

Taking it all in all, however, Courts are conservative, and their reasoning is usually sound. They are cautious, also, and venture into new and uncharted seas slowly, sounding by sounding. The exceptions prove the rule, and the engineer, in studying their decisions, will not run amiss if he takes the general results of their interpretation of the law as a safe guide. Courts and appraisers are both bound by the law. Whether it is such law as we would make ourselves is outside of the question; the law as we find it must govern us.

The Engineer in Valuation Work.

Although valuation work is based on law, and should adapt itself to the decisions of the Courts, yet the larger portion of the actual work of the valuer will be done in the capacity of the engineer, because almost all utility properties are essentially engineering projects, in which engineering experience, skill, and activity are predominant. The engineer, therefore, building soundly on the fundamental requirements of the law, may proceed to matters more properly within his own jurisdiction, where his accumulated practical experience with the laws of Nature and the constructive works of man will play the larger part.

Economics.

To the resources of the law and to the practical experience of the engineer must be added a third realm of information, which is, in many ways, a distinct domain by itself: that of finance or economics. Too often neither the engineering nor the legal mind is well trained in the great practical world of business, nor realizes the fluctuating value of money, nor is skilled in that class of problems which relates to the conservation of wealth, its limitations, use, and abuse. It is quite true that the more mature engineer should, and often does, know in a general way a good deal about finance, particularly if he has been much occupied with the broader aspects of engineering; that is to say, in those relationships where projects are financed and placed on a sound business basis. The great majority of engineers, however, do not come at all in contact with the preliminary operations under which projects are financed, and are often not well informed, and sometimes regrettably opinionated on matters of this kind. The nature and the duties of members of the legal profession are such that they are probably more often called into consultation as to the larger financial

questions than the great body of engineers, but even the practice of law does not always bring about a practical working knowledge of the ordinary rules that govern and guide the financier.

Utility Experts.

Taking advantage of the fact that there are many fields of knowledge in utility valuation which must be mastered in part, there has sprung into existence of late the term "public utility expert," obviously growing out of the desire to describe a composite calling, able to offer all the knowledge which must be possessed by a public utility valuer in all classes of valuation matters. There can be little question as to the futility of such enterprise.

It is impossible for any man to be a thoroughly competent expert in all phases of even one profession, and the work of valuation requires the co-operation of experts from several professions, and, as well, engineering specialists from each type of utilities to be valued. If there ever was a place where a "Jack of all Trades" is worse than useless, it is in valuation matters. The Courts and commissions are already overburdened by the necessity of listening to ambitious inexperience and voluble plausibility posing as experts, and the all-round expert, who can value anything under the light of the sun, should not be tolerated. If we are to have progress and justice, we should emphasize the importance of having co-operative effort from the most mature class of specialists in every line of specialized endeavor.

Value.

Turning now to some of the fundamental things which the law outlines as a foundation for all valuation processes, we may properly, first of all, inquire what value is. It would appear that this is a very simple question, but it is astonishing how few engineers, for instance, make any distinction at all between "value" and "cost." To grasp clearly the distinction between value and cost is one of the first fundamental principles which an engineering appraiser must learn. Very little reflection is needed to convince one that a thing may be worth much more than it cost, or it may be worth much less. It may even be worth more than it would cost new at the present time, or worth much less. This fundamental conception the legal profession and the economists almost always have well in mind, but it is often a

new thought to the engineer. This is because the engineer is largely dealing with new construction, in which cost most often approximates value.

Let us see what the Courts have said about so simple a thing as value:

VALUE DEFINED.

"Value is the relation of two services. The idea of value entered into the world for the first time when a man said to his brother, 'Do this for me, and I will do that for you.' They had come to an agreement. Then we could say the two services were worth each other." (State *v.* Yates, 10 Ohio, Dec. 150, 158; 19 Wkly. Law Bul., 150.)

"The word 'value' means the exchange power which one commodity or service has in relation to another." (State *v.* Yates, 10 Ohio, Dec. 150, 158; 19 Wkly. Law Bul., 150—citing Walker, "Science of Wealth.")

"The word 'value', in its commonly received signification, means the sum of money a thing will produce to the seller when sold. We are aware that this is not abstractly the true measure of value. The quantity of labor and capital necessary to produce a given article, or, in other words, the actual cost of its production, is the true criterion of its worth. For instance, if a manufacturer be asked the value of a yard of cloth, his opinion of value will be determined by his calculation of the expense of producing a yard of cloth of similar quality. If we ask a retail dealer the value of a yard of cloth, his opinion of its value will be determined by the quantity of money such an article will produce in the market. It is apparent that the marketable value may be affected by a multitude of circumstances which will not in their results extend to the cost of production.

"Value is in its nature so vague and indefinite that no human scrutiny can seize all its constituent parts, and, therefore, opinions of value are admissible in evidence from the necessity of the case." (Town of Rochester *v.* Town of Chester, 3 N. H., 349, 358.)

"The term 'value' in a constitutional provision that property shall be taxed according to value, means the worth of the property as compared with the money of the country—the standard by which all values are regulated." (City of Chattanooga *v.* Nashville, C. and St. L. R. Co., 75 Tenn. (7 Lea), 561, 569.)

"As market value. Bouvier, in his definition of value says: 'This term has two different meanings. It sometimes expresses the utility of an object, and sometimes the power of purchasing other articles with it. The first may be called the "value in use", and the latter the

"value in exchange." Webster recognizes a difference between extrinsic and exchangeable value. But in condemnation proceedings "value" will be held to mean market value.'" (Little Rock Junction Ry. *v.* Woodruff, 5 S. W., 792, 795; 49 Ark., 381; 4 Am. St. Rep., 51.)

"When applied to property, and no qualification is expressed or implied, 'value' means the price which the property could command in the market." (*In re* McGhee's Estate, 74 N. W., 695, 697; 105 Iowa, 9.)

"The word 'value', as defined by Bouvier, has two different meanings. It sometimes expresses the utility of an object, and sometimes the power of purchasing goods with it. The first may be called the 'value in use,' the latter, the 'value in exchange.' For the purpose of the law of eminent domain, the term 'value' has reference to the value in exchange, or market value. There are some cases which seem to hold that the value in use to an owner is to be taken if it exceeds the market value, but it will generally be found, on careful examination, that such cases either refer to the damages accruing to the owner from the taking, and not to the value of the property itself, or they overlook the distinction between the two things. The consensus of the best-considered cases is that for the purpose in hand the value to be taken is the market value." (San Diego Land and Town Co. *v.* Neale, 20 Pac., 372, 374; 78 Cal., 63; 3 L. R. A., 83.)

"The primary meaning of 'value' is worth; and this worth is made up of the useful or estimable qualities of the thing. Ordinarily, when an article of sale is in the market, and has a market value, there is no difference between its value and the market price, and the law adopts the latter as the proper evidence of value. This is not, however, because 'value' and 'price' are really convertible terms, but only because they are ordinarily so in a fair market." (Kountz *v.* Kirkpatrick, 72 Pa. (22 P. F. Smith), 376, 386; 13 Am. Rep., 687.)

"Value of Railroad. The value of a railroad track at any given time is not the original cost nor its ultimate cost after years of expenditure in repairs and improvements. On the other hand, its value cannot be determined by ascertaining the value of the land included in the roadway, assessed at the market price of adjacent lands, and adding the value of the cross-ties, rails, and spikes. The value of land depends largely upon the use to which it can be put, and the character of improvements upon it. The assessable value for taxation of a railroad track can only be determined by looking at the elements on which the financial condition of the company depends;—its traffic as evidenced by the rolling stock and gross earnings in connection with its capital stock." (Pittsburg, C. C. and St. L. Ry. Co. *v.* Backus, 14 Sup. Ct., 1114, 1118; 154 U. S., 421; 38 L. Ed., 1031.)

"The value of a railroad for purposes of taxation is not the mere value of its right of way, roadbed, its structure, its depot ground and structures thereon, considered by themselves, but the value of all these as an operating going concern; this value being in general determinable by the profits which result from its operation." (State *v.* Austin and N. W. R. Co., 62 S. W., 1050; 94 Tex., 530.)

It must be fully realized that, in public utility valuation, we are endeavoring to ascertain the fair value as between the public, on the one hand, with its needs for the service, and the owners of the property which has been devoted to the public use, and is practically incapable of extricating itself from that service, and, therefore, peculiarly dependent on just and fair treatment for its conservation.

Property.

Another of the fundamental concepts which should be well established in the mind of utility commissions and engineers is the meaning of the word "property." Here, again, it seems strange that any considerable difference of opinion should arise about a definition which has been so long a subject of inquiry as that of the word "property," and yet lack of a clear idea as to what is being appraised in a public utility is mainly responsible to-day for the wide divergence of views as to how to proceed and what to value.

One might well ask how it is possible to accomplish anything without preliminary agreement as to so fundamental a proposition. It must be conceded that it is fundamental, in valuation work, to know what it is we are to value, yet engineers and appraisers generally will pass out opinions as to the value of plants without any intelligent inquiry as to the fundamental conception of what they are valuing. This is where the law and the Courts lay a foundation that ought to be clearly in mind before one tries to discuss details or proceed to estimate.

There arise, from the prevailing fundamental misconceptions, two opposing schools of thought, born on opposite sides at interest, which, being radically at variance with each other, can, of course, find no common ground for agreement in detailed methods.

On one side is ranged a school of thought, consisting, for the most part, of the majority of those advocates of municipal ownership and operation who have first approached the subject largely from that side of the question. This school of thought, as well as

certain of the more recently formed utility commissions and their staffs, who have been influenced by such arguments, conceive that the utility should be examined largely from the standpoint of the actual cost, or treated as a past investment, and that it can be considered to have value only in so far as actual cash or money has been put into the plant and property. The common viewpoint of this class of appraisers is that this past cost should be stripped of all accretions and increments of growth that are not represented at some time or other by actual expenditures. Certain extremists of this class of thinkers would even strip the actual "past investment" of its appreciation in the value of land and other like appreciations, while confining value to the actual past cash cost. They would further deduct from it any depreciation that might be found to have accrued.

On the other hand, as an opposing school of thought, there has developed a class of valuers who hold that the plant and business of a utility company together constitute a "property", which may have accretions in value outside of actual cash investment, and that such property may appreciate through such accretions, just as it may depreciate through age, lack of adaptability, or other causes.

Now this is a very fundamental and important difference of opinion, and needs intelligent and fair-minded investigation before we are at all competent to enter into detailed questions of valuation. Are we to find the actual dollars that have been invested in a plant and business, or are we to find the value of the resulting "property"? What does it profit us to discuss details when this serious and fundamental difference exists? The two schools of thought that have approached this subject of valuation differ so widely on this question that it is safe to say that much more than one-half of all our time in Courts and before commissions is taken up with testimony and argument directed to the one or the other of these two theories. Most of the recent literature on the subject of valuation work bears evidence of a fatal disagreement on this one fundamental point.

Curiously enough, this important question is emphatically settled by the highest authority we have, or can hope to have: the Constitution of our country as interpreted by the Supreme Court of the United States.

Property is a well-defined word in the meaning of the law, and the Fourteenth Amendment to the Constitution of the United States

is fundamentally the authority under which the Courts proceed in the determination of fair value of property for use in the fixing of rates or the ascertainment of value for condemnation or sale. It reads, in part, as follows:

"Nor shall any State deprive any person of life, liberty, or property without due process of law."

The provision of the Fifth Amendment to the Constitution, controlling condemnation cases by the United States, reads as follows:

"Nor shall private property be taken for public use without just compensation."

In the Constitutions of the States are similar provisions in which the word "property" is used. The significant thing is, that whether we look to the Fourteenth or to the Fifth Amendment, or to the State Constitutions, what shall not be taken is "property." The Fifth Amendment prohibits the United States from taking property without compensation. The Fourteenth Amendment prohibits any State from taking property without due process of law.

Nor has there been left undecided the question, which property devoted to the public use shall be treated differently in respect to its ownership and right of protection from other private property, as will be shown by later quotations. What is here to be emphasized is that the fundamental law of our land does not use the terms "investment", or "cash cost", or "past cost", or "actual expenditure", or any one of the numberless approaches to actual cash investment that would be used if a property were not precisely meant, with all that the term involves.

However, it is now proper to inquire further: What is property? Let us look for a moment at some of the more common legal definitions.

Property Defined.

"The sole and despotic dominion which one man claims and exercises over the external things of the world in total exclusion of the right of any other individual in the universe." (2 Bla. Com., 2.) "The right to possess, use, enjoy, and dispose of a thing (56 N. Y., 268) which is in itself valuable (4 McLean, 603). The right or interest which one has in lands or chattels. (6 Binn., 94; 4 Pet., 511.) The free use and enjoyment by a person of all his acquisition without any control or diminution, save only by the law of the land. (2 Ark., 291; 61 Hun, 571.) The right of a person over a thing (*in rem*) indefinite in point of user." (Austin's Lectures.)

"Webster defines 'property' to be the exclusive right of possessing, enjoying, and disposing of a thing; ownership; an estate, whether in lands, goods or money." (Spring Valley Water-works *v.* Schottler, 62 Cal., 69, 72, 84. See also Banning *v.* Sibley, 3 Minn., 389, 404 (Gil., 282); McKeon *v.* Bisby, 9 Cal., 137, 142; 70 Am. Dec., 642.)

"The term 'property', as used in the Constitution, is not confined to tangible objects which can be passed from hand to hand, but includes those rights of possession, disposal, management, and of contracting with reference thereto, which render property useful, valuable, and a source of happiness, the right to the pursuit of which is preserved." (State *v.* Kreutzberg, 90 N. W., 1098, 1100; 114 Wis., 530; 58 L. R. A., 748; 91 Am. St. Rep., 934.)

"The term 'property', as used in the Constitution, Articles 1 and 17, forbidding the taking of property for public use without adequate compensation being made therefor, means not only the thing owned, but also every right which accompanies ownership and is its incident." (Gulf, C. and S. F. Ry. Co. *v.* Fuller, 63 Tex., 467, 469; Ft. Worth and R. G. Ry. Co. *v.* Jennings, 13 S. W., 270; 76 Tex., 373; 8 L. R. A., 180.)

"Anticipated Profits. Anticipated profits are not, and cannot be, held and regarded as property in the ownership or possession of him who owns the article out of which profits are expected to flow. The property is one thing, and remains untouched. The profits are *in esse,* and cannot be claimed as property." (Munn *v.* People, 69 Ill., 80, 91.)

"The term 'property', in its broad sense, includes even a franchise, so the word 'property' as used in the statute providing for the taxation of all property not specifically exempted, includes a franchise, and hence a tax on a turnpike is property." (Frankfort, L. and N. Turnpike Road Co. *v.* Commonwealth, 6 Ky. Law Rep., 391, 392.)

"Property in its broadest and most comprehensive sense, includes all rights and interest in real and personal property, and in easements, franchises, and in corporeal hereditaments. That which may be taken for public uses is not exclusively tangible property. Wherever the right of eminent domain exists, whatever exists in any form, whether tangible or intangible, may be subjected to the exercise of this power, and may be appropriated to public use when necessity demands. A franchise is nevertheless a right in the nature of 'property' within the sense that the word is used in the eminent domain law, and no reason is perceived why it could not be condemned for the use of the public. We know of no policy of the law that would forbid the taking of such property for public uses." (Metropolitan City R. Co. *v.* Chicago W. D. Ry. Co., 87 Ill., 317, 324.)

"Water. Water, when reduced to possession, is property, and it may be bought and sold, and have a market value; but it must be in actual possession, subject to control and management. Running water in a natural stream is not property, and never was. The construction of a dam across an outlet of a lake or a river or a creek for the purpose of securing power with which to operate mills or factories is not a reducing of the water to possession, or to control or management, in such a sense as to change its legal character and make it property. If it did, the owner at times might find it difficult to control, especially in case of freshets or floods. He might have trouble in preventing it from being precipitated upon the lands of his neighbors below, and find it unpleasant to respond to them for the damages caused by his property." (City of Syracuse *v.* Stacey, 62 N. E., 354, 355; 169 N. Y., 231.)

It seems to be apparent from these definitions that a property, even in a public utility, is not a mere aggregate of separate structural materials, priced as would be the stock of a shopkeeper, nor can such property be properly confined to a past investment which must be traced out, dollar by dollar, to show how much actual cash has really been put into it. A property, even in a public utility, is a right of possession in a complete entity actively engaged as a co-ordinate whole. This property, as a whole, lives through the ordinary vicissitudes of life, and may have its accretions of value, due to the growth, expansion, and enterprise of the adjacent population, as well as its depreciations, due, among other things, to lessening demand, structure disuse, or changes in the arts. These may be outlined generally, as follows:

ORIGINAL COST.

Depreciations.		Appreciations.
(1) Wear and tear on machinery and plant. Decay.....	Period of growth and usefulness, or of decline and abandonment.	(1) Growth of population and demand on original installation.
(2) Decreased usefulness of plant due to shifting population...		(2) Enlargements and extensions.
(3) Changes in general demand producing obsolescence.....		(3) Growth of population on extensions.
(4) Improvements in the art, requiring replacement.....		(4) Growth of habit, increasing use of service per capita.
(5) Changes required in the source or method of manufacture...		(5) Increasing cost of replacement.
(6) Insufficiency of original installation by reason of increased population.....		(6) Increasing value of real estate.
(7) Checks to growth of population or surrounding industries....		(7) Increasing cost of materials and labor.
(8) Decrease in price of labor and materials.....		(8) Increased business due to enterprising management.
(9) Increased competition.....		(9) Development of new uses for supply or service.
(10) Imprudent expenditures.....		(10) Economies in production effected by good management.

PRESENT VALUE.

A property, therefore, is like an individual. It is born; it goes through its youthful training; it encounters its vicissitudes; it comes to its maturity, or period of usefulness; it expands with the community perhaps; it has its period of decline, and is supplanted by better methods of service or production; eventually it will die, at least so history teaches us. Constantly, however, we find valuers (usually representing what may be called the buyer side of the question) who, through mistaken sympathy, leading to unconscious unfairness, or through ignorance of all sides of the question, or more rarely, it must be regretfully said, through direct prejudice or deliberate intent, would deprive a utility property of its natural growth in value, on the theory that they are dealing here with something other than private property, devoted to the public use. In other words, they would deal with private property devoted to public use, as property the natural growth of which could be checked, or portions of the value of which could be appropriated to the public ownership because of its public use.

Of course, it would be a very interesting and perhaps even profitable inquiry to discuss the question as to whether the Constitution of the United States, as construed by the higher Courts, may be wrong in its theory of property. It might also be of great interest and profit to inquire whether interest is moral, and to raise many other fundamental questions of a like nature. Many intelligent people can be found (and some of them are in valuation work), who would like to see the Constitution revised, at least so far as relates to private property devoted to public use. Such people would not dream of appropriating private property for private use, but are apparently quite willing to appropriate, in part at least, private property devoted to the public use.

The right to hold property, with all its increment of growth, is fundamental to our civilization. Without this right our civilization would vanish. It is to the public interest that this should continue to be so, for the moment we begin to treat private property, devoted to the public use, as public property, then private property will cease to be available for the public uses and the needs of the people, to the detriment of every one. What we are properly striving for, in all the recent movement toward regulation of private property devoted to public use, is not the lessening of the value of that property, but the

correction of abuses and evils which grow out of its improper use or abuse. Wherever, then, we find wide differences of opinion as to the value of a property, devoted to the public use, we will usually find that a large part of the difference is due to differing conceptions as to what it is that is being valued, and differing definitions resulting from that fundamental departure.

If appraisers, and especially engineers, could once get it thoroughly understood that it is not their function to say what is to be appraised—that function belongs to the law, and such points are fundamentally determined by the Constitution of the United States, as interpreted by the Supreme Court—then most of our troubles and confusion in appraisal work would be over.

The Right of Regulation.

The leading case in the Supreme Court of the United States upholding the power to regulate the rates of public utilities was handed down as far back as 1876, and is based on the proposition that one who devotes his property to a public use:

"In effect grants to the public an interest in that use, and must submit to be controlled by the public for the common good to the extent of the interest he had thus created.

"He may withdraw his grant by discontinuing the use, but so long as he maintains the use he must submit to its control." (Munn *vs.* Illinois, 94 U. S., 43, 24th Law Ed.)

It is interesting to examine historically the widening acceptance which has been given to this principle, even by the Courts themselves. Little by little it has been extended from pure monopolies into the domain of quasi-monopolies, and finally to public utilities that are competitive, such as lighting, heat, and power; so that now it is virtually conceded that all business in which public grants are necessary is subject to public regulation. It is now coming to be well understood that a successful public utility property, to give good service, should, as a rule, be a monopoly. The old idea which prevailed a generation ago, and was recognized by the laws of such States as California—that it was a good thing to have rivalry in public service corporations—has passed, and we recognize that a regulated monopoly is, all things considered, by far the best method of conserving our resources and serving ourselves.

When, however, we come to value a monopoly as a property, and especially where a monopoly is subjected to rate regulation, we appear to come to a difficulty that its value may depend, to a large extent, on the rates, and as the proper rates are obviously dependent in part on the value, we seem at first glance to be reasoning in a circle. This difficulty, however, is not as great as it appears, because on closer investigation we shall find that, when a property consists largely of a physical plant, we can find a proper value for the property, even though its value depends in part on a regulated rate of return. This is also possible where the rates must afford a fair return, which rate of return, however, must be assumed in advance. In other words, where there is a fixed physical plant and a fixed fair rate of return, a definite value results.

In valuing a regulated monopoly or partial monopoly, we cannot, of course, proceed as we would with an ordinary business, fluctuating with its good-will, nor can we properly capitalize existing income independent of physical plant. We are, therefore, evidently limited to methods in which value is not wholly dependent on income, because the income is to be regulated. We are usually obliged, therefore, to reason from the basis of the physical property, plus the cost of financing it into proper and remunerative income.

A great deal of exception has been taken to this embarrassing condition, because it at once does away with a most common and natural method of arriving at value, that of capitalizing income. It is difficult, however, to see how this can be overcome, so long as rate regulation is a fundamental precedent to a valuation.

The Supreme Court of Wisconsin, in the recent Appleton Case (142 Northwestern Reporter, 476, May 31st, 1913), evidently feeling the embarrassment of the situation, says:

"The commercial value of the business in full operation and entitled to charge reasonable rates for its service must, however, be considered as approximating the compensation which should be allowed for the property; in other words, the sum which the business should be capitalized for, in order that the owner should receive a reasonable return on the investment when the business is conducted with reasonable business skill, and charges such reasonable rates for service as the law permits."

This rule would bring permissible net income, or, as the English authorities put it, "maintained and maintainable net income," into a

position of prominence at once, were it possible to define maintained and maintainable income, or, "such reasonable rates for service as the law permits," entirely aside from all consideration of the property devoted to the public use itself, but the Wisconsin Supreme Court nowhere else in its opinion gives us any clue as to how this "reasonable rate" is to be defined, and ends by approving the findings of the Commission, based on examination of the physical property in connection with other evidences of value.

There seems to be no escape from the conclusion that, so long as rates of return are to be regulated, they cannot be used as a sole foundation for ascertaining value. Some other and more devious path is forced on our attention, in which the importance of the return is, if not altogether eliminated, at least so minimized as to lose its preponderating influence.

The Starting Point.

An examination of their decisions will show that the Courts have reached a definite conclusion that the proper interpretation of the Constitution requires them to insist on two essentials:

(*a*) That what is to be valued is the "property."

(*b*) That what is to be ascertained is the present value of this property.

This has compelled the Courts to require such an investigation as will present to them all the factors necessary for the determination of the present value of the property devoted to the public use.

The following is an extract from Minnesota Rate Cases, 230 U. S., 352, 434:

"(1). The basis of calculation is the 'fair value of the property' used for the convenience of the public. Smyth *v.* Ames, *supra* (p. 546). Or, as it was put in San Diego Land and Town Co. *v.* National City, *supra* (p. 757), 'What the company is entitled to demand, in order that it may have just compensation, is a fair return upon the reasonable value of the property at the time it is being used for the public.' See also San Diego Land and Town Co. *v.* Jasper, *supra;* Willcox *v.* Consolidated Gas Co., *supra.*

"(2). The ascertainment of that value is not controlled by artificial rules. It is not a matter of formulas, but there must be a reasonable judgment having its basis in a proper consideration of all relevant facts. The scope of the inquiry was thus broadly described in Smyth *v.* Ames, *supra* (pp. 546-547): 'In order to ascertain that value, the

original cost of construction, the amount expended in permanent improvements, the amount and market value of its bonds and stock, the present as compared with the original cost of construction, the probable earning capacity of the property under particular rates prescribed by statute, and the sum required to meet operating expenses, are all matters for consideration, and are to be given such weight as may be just and right in each case. We do not say that there may not be other matters to be regarded in estimating the value of the property. What the company is entitled to ask is a fair return upon the value of that which it employs for the public convenience. On the other hand, what the public is entitled to demand is, that no more be exacted from it for the use of a public highway than the services rendered by it are reasonably worth.'"

Ames *vs.* U. P. Railway, 64 Fed., 165.

San Diego Land and Town Co. *vs.* National City, 74 Fed., 74 (189) (Ross): "In my judgment it is the actual value of the property at the time the rates are to be fixed that should form the basis upon which to compute just rates, having at the same time due regard to the rights of the public and to the cost of the maintenance of the plant and its depreciation by wear and tear."

Cotting *vs.* K. C. Stock Yards (1897), 82 Fed., 850, 854. "If improvements have been made in the vicinity of the property, the growth of the city or town where it is located, the building of railroads, the development of the surrounding country, and other like causes give property an increased value, the owner cannot be deprived of such increase by legislative action which prevents him from realizing an income commensurate with the enhanced value of his property."

Smyth *vs.* Ames, 1898, 169 U. S., 466; 18 Sup. Ct., 418 (Justice Harlan). "We hold, however, that the basis of all calculations as to the reasonableness of rates * * * must be the fair value of the property being used by it for the convenience of the public."

Willcox *vs.* Consolidated Gas Co., 212 U. S., 19-52 (1909); 29 Sup. Ct., 192-53.

This last case is an interesting one, as it applies this line of reasoning to some very extreme conditions, and it is a recent case. It may be well to quote from the Circuit Court (157 Fed., 849), from which the appeal was taken, more fully, as follows:

"It is impossible to observe this continued use of the present tense in these decisions of the highest court without feeling that the actual or reproductive value at the time of inquiry is the first and most important figure to be ascertained, and these views are amplified by San Diego Land Co. *v.* Jasper (C. C.), 110 Fed., at page 714, and Cotting *v.* Kansas City Stock Yards (C. C.), 82 Fed., at page 854,

where the subject is more fully discussed. Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the one at bar. The complainant by itself and some of its constituent companies has been continuously engaged in the gas business since 1823. A part of the land in question has been employed in that business for more than two generations, during which time the value of land upon Manhattan Island has increased even more rapidly than its population. So likewise the construction expense, not only of buildings, but of pipe systems under streets now consisting of continuous sheets of asphalt over granite, has enormously advanced.

"The value of the investment of any manufacturer in plant, factory, or goods, or all three, is what his possessions would sell for upon a fair transfer from a willing vendor to a willing buyer, and it can make no difference that such value is affected by the efforts of himself or others, by whim or fashion, or (what is really the same thing) by the advance of land values in the opinion of the buying public. It is equally immaterial that such value is affected by difficulties of reproduction. If it be true that a pipe line under the New York of 1907 is worth more than was a pipe line under the city of 1827, then the owner thereof owns that value, and that such advance arose wholly or partly from difficulties of duplication created by the city itself is a matter of no moment. Indeed, the causes of either appreciation or depreciation are alike unimportant, if the fact of value be conceded or proved; but that ultimate inquiry is oftentimes so difficult that original cost and reason for changes in value become legitimate subjects of investigation, as checks upon expert estimates or bookkeeping inaccurate and perhaps intentionally misleading. *Cf.* Ames *v.* Union Pacific R. R. (C. C.), 64 Fed., at pages 178, 179. If 50 years ago, by the payment of certain money, one acquired a factory and the land appurtenant thereto, and continues to-day his original business therein, his investment is the factory and the land, not the money originally paid; and unless his business shows a return equivalent to what land and building, or land alone, would give if devoted to other purposes (having due regard to cost of change), that man is engaged in a losing venture, and is not receiving a fair return from his investment, *i. e.*, the land and building. The so-called 'money value' of real or personal property is but a conveniently short method of expressing present potential usefulness, and 'investment' becomes meaningless if construed to mean what the thing invested in cost generations ago. Property, whether real or personal, is only valuable when useful. Its usefulness commonly depends on the business purposes to which it is or may be applied. Such business is a living thing, and may flourish

or wither, appreciate or depreciate; but, whatever happens, its present usefulness, expressed in financial terms, must be its value.

"As applied to a private merchant or manufacturer, the foregoing would seem elementary; but some difference is alleged to exist where the manufacturer transacts his business only by governmental license —whether called a franchise or by another name. A license, however, cannot change an economic law, unless a different rule be prescribed by the terms of the license, which is sometimes done. No such unusual condition exists here, and, in the absence thereof, it is not to be inferred that any American government intended, when granting a franchise, not only to regulate the business transacted thereunder, and reasonably to limit the profits thereof, but to prevent the valuation of purely private property in the ordinary economic manner, and the property now under consideration is as much the private property of this complainant as are the belongings of any private citizen. Nor can it be inferred that such government intended to deny the application of economic laws to valuation of increments earned or unearned, while insisting upon the usual results thereof in the case of equally unearned, and possibly unmerited, depreciation.

"I think the method of valuation applied by the report to land, plant, mains, services, and meters lawful."

On appeal to the United States Supreme Court, the above position of District Judge Hough was approved. Justice Peckham, in delivering the opinion of the Court, says (212 U. S., 19, 52; 29 Sup. Ct., 192; 53 L. ed., 382, January 4th, 1909):

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property, which legally enters into the consideration of the question of rates, has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule. We do not say there may not possibly be an exception to it, where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such increased value unjust to the public. How such facts should be treated is not a question now before us, as this case does not present it. We refer to the matter only for the purpose of stating that the decision herein does not prevent an inquiry into the question when, if ever, it should be necessarily presented."

It is apparent, from these opinions, that there is no escape from the conclusion that in all ordinary cases we must value a public utility as a "property" and find its value as of to-day. Nothing short of a reversal of its own opinion by the Supreme Court, a new Consti-

tution for the United States, a revolution, or what would be (in the writer's opinion at least) infinitely worse, the recall of judicial opinion, could alter this present status.

If, therefore, we must value a utility as a property, and value it now, it is quite obvious why the Courts have concluded that reproduction cost, less depreciation, of the existing property (or, as we will call it for the sake of brevity, "reproduction") is usually a leading and most important line of evidence to aid in arriving at value. The Courts do not suggest the neglect of any other lines of evidence, but, under normal circumstances, they place weight on such evidence as tends to show the value of the property now.

The general trend of recent decisions has been to make reproduction cost methods a controlling and important line of evidence as to value under normal conditions.

In a Minnesota rate case—Steenerson *vs.* Great Northern Railway, 69 Minn., 353, October 20th, 1897—the Court said:

"Then the burden is on the railroad company to show that the rates fixed by the Commission are unreasonable, and for this purpose the original cost of the road, the amount of its present fixed charges, and its history, are material only so far as they show what it would now cost to reproduce the railroad."

In 1911, the later Minnesota rate cases, Shepard *vs.* Northern Pacific Railway Co., 184 Fed., 765, April 8th, 1911:

"The master rightly decided that the cost of reproducing this property new was a more rational and reliable measure of its real value than the original cost of its acquisition and construction or the market values of the stocks and bonds of the companies, and upon that basis he made his findings."

In San Diego Land and Township Co. *vs.* National City, 74 Fed., 79, May 4th, 1896, the Court says it is "Present value", and, on appeal, Justice Harlan refers to "reasonable value of the property at the time it is being used for the public." (174 U. S., 739.)

In the Consolidated Gas Case *vs.* the City of New York, 157 Fed., 849, Judge Hough said:

"In every instance, however, the value assigned in the report is what it would cost presently to reproduce each item of property in its present condition, and capable of giving service neither better nor worse than it now does. As to all the items enumerated, therefore, from real estate to meters, inclusive, the complainant demands a fair

return upon the reproductive value thereof, which is the same thing as the present value, properly considered. To vary the statement: complainant's arrangements for manufacturing and distributing gas are reported to be worth the amounts above tabulated if disposed of (in commercial parlance) 'as they are.'

"Upon authority I consider this method of valuation correct. What the court should ascertain is 'fair value' of the property being used (Smyth *vs.* Ames, 169 U. S., 546); the 'present' as compared with 'original' cost; what complainant 'employs for the public convenience' (169 U. S., at page 547); and it is also the value of the property at the time it is being used (San Diego Land Co. *vs.* National City, 174)."

The Courts are extremely cautious not to commit themselves to reproduction methods, as a rule, for finding value, and this is because they see clearly that exceptional conditions might make the evidence of reproduction methods at times misleading and dangerous. As a matter of practical inquiry, however, under the conditions as ordinarily met with, and in the great majority of valuation cases, the Courts pay special attention to reproduction methods, and the largest portion of the testimony in such cases is usually directed to this line of evidence, for the very good reason that, unless abnormal conditions are present, it is the method which will most nearly disclose the value of the property as of to-day.

Let us inquire further what general procedure we may adopt in valuation work to be in harmony with the reasoning of the Courts on this subject.

Practical Methods of Utility Valuation.

The function of valuing a public utility property consists essentially of two distinct operations:

First.—The collection of all the facts and evidence tending to throw light on the question of value; and

Second.—The judicial function of weighing the evidence, apportioning to each line of evidence its relative degree of importance, and reasoning from this summation to the broader question of value.

In valuation work, much confusion of thought arises from the very human anxiety to reason on the broader question of value before all the data are accumulated, properly digested, and placed in their proper perspective.

The facts and evidence desirable for a full understanding of the value of a utility property are generally as follows:

First.—The cost to replace the property at the present time as a going concern.

This line of investigation requires:

(*A*) An inventory of the present existing physical property, used and useful for the purpose of the utility.

(*B*) A statement of its earnings and expenses for a preceding series of years, not less than five, and preferably ten or more.

(*C*) Computation of the cost to replace the property now, and reproduce its income as a going concern, all in a way which is humanly possible.

(*D*) An estimate of the loss of value in the existing property, by reason of age, changing requirements, advances in the arts, or use.

Second.—The market value of the securities of a property, where a broad and representative market for such securities can be said to exist.

Third.—The value of other properties similarly situated, where such value has been carefully and fully determined. This can be used only as a general check, however.

Fourth.—The conditions which surround the particular property to be valued, such as the character of population, its past growth and present business enterprise; the presence or absence of judicious management of the utility property, and its conservative extension; freedom of the property from mishaps; the value of the service to the individual consumer; competition, if any, and possible future contingencies arising both from changes in public demand or substitution of other service more acceptable to the population; and, finally, the legal status of the property, and attitude of the public and its representatives toward it.

Fifth.—The Courts hold that, where past cost can be obtained, it is competent evidence. The fact that it is competent does not mean that it is controlling, and does not indicate the weight it should have. What weight it should have in

any case depends on whether or not the conditions when the past cost was incurred are the same as the conditions at the date of the valuation. If they are, it is valuable evidence. In proportion as the conditions differ, the weight to which this testimony is entitled grows less. It is for this reason that recent costs, which presuppose unchanged conditions, are treated as valuable factors, while ancient costs are usually valueless.

The judicial reasoning which should follow the compilation of these lines of inquiry is as follows:

First.—To be assured that the evidence is without material error.

Second.—To determine the amount of credence to give to the witnesses who introduce evidence. This may be based largely on their practical experience in the matters in hand, as well as their conscientiousness and freedom from bias.

Third.—To determine that each independent line of evidence introduced is in itself logical and complete, and is not confused with any other line of evidence, such as the mixing up of past cost with present cost to reproduce, etc.

Fourth.—To decide what weight should be given to the various lines of evidence introduced.

Fifth.—To determine in each case the extent to which each line of evidence is a material factor in measuring value, and if it is not such a factor, to reason why not.

Sixth.—To determine in particular whether the cost of reproduction, including the cost of reproducing the present income of the company, is or is not the most important factor in reaching a sound conclusion as to value.

Seventh.—To conclude finally on the value, from a broad and adequate review of all the facts, with an open mind, in a judicial manner, and as a matter of common sense.

Eighth.—To check the value so obtained by such lines of inquiry as will properly be of aid.

In weighing evidence as to value, it is important to remember:

First.—That value is, after all, a matter of intelligent opinion, based on all the ascertainable facts with special reference to the conditions of the case in hand.

Second.—That cost is not usually coincident with value.

Third.—That the Courts uniformly have insisted that it is the present value, or value now, that must control.

Fourth.—That reproduction is usually the most direct line of evidence bearing on value now.

Fifth.—That cost now is not necessarily value now, for it is easily conceived that, under many circumstances, a plant may be worth more or less than even the cost to reproduce it now.

Sixth.—That the weight to be given the evidence of individual valuers is in direct proportion:

(*a*) To the extent of their direct past practical experience;

(*b*) To their breadth of view and study in callings other than their own profession affecting valuation work;

(*c*) To their character and standing as fair and unbiased men, used to considering both sides of the questions at issue in an impartial manner.

Reproduction.

The foregoing discussion, quoted opinions, and outline of valuation procedure, demonstrate why reproduction has assumed its importance in public utility valuation. It has not been originated by engineers to torment the Courts, but it has been concluded by the reasoning of the Courts to be a logical and important line of evidence, and the Courts have compelled the Engineering Profession to work out its details in a practical way, and in a manner that is humanly possible.

In considering reproduction, however, we are confronted, first of all, by the fact that the more fully it is examined the more it is seen to be a hypothetical proceeding, using this term in its legal sense, Nothing about it is so real as the results to the owner. The more it is studied the more it must be conceded that reproduction is purely a conceptual forecast in a most difficult and specialized art—that of financing and building especially designed public works for distinct kinds of public service. Even the art of constructing public service plants is comparatively new; the number of engineers specially skilled in the broader aspects of such arts is relatively few; and the mature and experienced engineers in such specialties are fewer still. It is quite obvious that in the immature thought of impractical and in-

experienced minds, reproduction can be made absurd to the point of ridicule, and, even in the hands of an experienced, painstaking, and unbiased investigator, it is questionable whether the full, proper, and natural sequence of events necessary to the hypothetical rebuilding of visible property can always be properly and intelligently thought out with any reasonable degree of thoroughness and proper prophecy, without omission of matters which would probably unexpectedly occur in any actual procession of events.

Conceptual Processes.

Let us see what it is that the Courts have asked us to do when we are made to undertake the task of reproducing. Having familiarized ourselves with the existing property which is to be valued, we must, in imagination, as a prolonged conceptual process, retrace, step by step, every feature of the procedure that would have to be taken if that property were completely wiped out, and we were responsible for the task of entirely rebuilding it in a reasonable length of time and in a manner which is humanly possible. Should we unconsciously omit one important step necessary to that rebuilding, we will fail to reach the full cost. Being in possession of retrospect, we are constantly subject to the temptation to substitute an easy and virtuous hindsight for that real lack of foresight which would be our condition in actual life and under ordinary conditions. If we do full justice to the procedure, we must trace two lines of human endeavor, that is, the mental labor as well as the physical labor which produced the result before us. Mental endeavor will include the development of the project and the preliminary steps of promotion. This must include the necessary time and pains that would naturally be taken to see that the project is practical, the time and attention necessary to negotiate the franchise, investigate the methods of service, and estimate the preliminary work. We must at every step of the way outline to ourselves the necessary lengths of interlapsing time to do all these things, as well as the amount of labor to design, contract for, and actually build each structure. Much of our cost will be entirely dependent on the length of time we assume for these practical operations. It is quite certain that the most trained imagination, practically acquainted with similar construction, will not always entirely succeed in redeveloping every detail involving expense

in such a procedure. The Courts, in advocating reproduction as a valuable line of inquiry, have probably thought that a hypothetical reconstruction was simple. At first glance it would appear simple; so many pounds of rail at so much, so many ties at so much, and total so much. Any one coming to the task of reproducing a property in this spirit has singularly misconceived the extent and complexity of the problem before him.

The Sidewalk Illustration.

The fallacious attitude of many estimators can best be shown by a simple illustration of sidewalk construction. You sit in your library window with your little boy and watch the construction of a sidewalk across the street, and you ask your boy how sidewalks are built. In all probability he will reply, with his eye on the scene, "Why, they take crushed stone and cinders and sand and cement and mix them up and spread it out to dry between planks that keep the edges straight; then they tamp it all down, smooth it over, leave it to dry, and when it is done they clean up and go away."

Now, this is really about all the average grown person sees of engineering work. Even some judges and economists would hardly give a fuller answer. They might add that the sidewalk costs a certain sum per square foot, because the sidewalk contractor received that sum, and that estimate of cost would be the general impression of a good many mature people as to the cost of sidewalks.

Let us analyze the matter a little further. Assume that you are familiar with municipal engineering, and you will find in your mind's eye vastly more things that have to be done than your boy could see. A petition from the property owners has to be presented to the common council of the city, with the signatures of the majority of the frontage owners attached thereto. The council perhaps sends an inspector, who reports on the matter. A committee of the council deliberates and reports back favorably to the council. An ordinance has to be drawn by the law department of the city, the engineering department submits an estimate for the cost of the work, and the council then passes the ordinance. A warrant is issued from the proper court for the spreading of the assessment and commissioners are appointed who apportion the tax and make their report. A hearing is held at which objectors may appear. The assessment is perhaps

finally confirmed, and a warrant issued to the county or city collector, the tax bills are sent out, and the moneys are received by the treasurer.

In the meantime, the city engineering department has drawn up specifications for the work, describing minutely just what is to be done, the kind of material to be used, and its method of mixing and laying. Notices are inserted in the newspapers calling for bids. Contractors make their estimates and appear at the letting; the bids are received, opened, and tabulated; a report is made by the engineer to the council; and the work is thereupon let to the lowest bidder. A contract is drawn up by the law department, bonds of a proper amount are deposited, and the contract is signed. In the meanwhile the city engineer sends an assistant who stakes out the established grade which had heretofore been fixed by the council from surveys of the city by the engineering department. An inspector is appointed to see that the specifications and contract are complied with. In the meantime the contractor had figured out the necessary quantities of material, purchased the proper quality, and had hauled it to the work. He then engaged a mason and helpers, a foreman and laborers, and then—your little boy saw them appear on the street to do the work.

Now, what was the real cost of that sidewalk? Certainly the city treasurer's books do not show it. Did the engineer in charge know? Did the contractor know? Did your little boy know? Did any one ever really know?

Public Misconceptions.

So it is with the average public utility valuation. The public thinks of it as a list of visible material which must simply be priced, and when the engineer talks about preliminary expense, length of time, methods of construction, plant financing, overhead charges, developing the business, and the like, the public thinks he is trying to inflate the valuation by what it likes to call "intangible" items.

The fact is, as has recently been well emphasized by M. E. Cooley, M. Am. Soc. C. E., in an address before the Annual Meeting of the Western Society of Engineers,* that neither the Courts nor the public have any clear idea of the real cost of building up a property, and it is a public duty for the Engineering Profession to point out all the necessary elements of cost which would be incurred in a real enter-

* *Journal*, Western Soc. of Engrs., January, 1914, p. 1.

prise, and to insist that hypothetical reproduction estimates must be made in a manner which is humanly possible and practically feasible, such as would be the case in real life and in line with real practical experience.

Most attempts at hypothetical reproduction produce estimates that arrive at a less cost of reproduction than would be found necessary in real actual building, by reason of the before mentioned tendency to omit minor items, overlook necessary auxiliaries, or fail to appreciate all the contingencies that are likely to occur in an actual undertaking. All engineers realize that the less we know about a given line of building work, the more apt we are to underestimate it, and almost every one who has built a house can recall how much they underestimated the final cost, and why.

Reproducing the Income.

When we reproduce a property, we must reproduce all of the property there is; that is, we must reproduce it up to its present state of efficiency, including the cost of financing the plant to the point where it has income and is paying a fair return. This is so astonishingly simple as a proposition that one constantly wonders why it should be disputed, yet, as a matter of fact, few appraisers have attempted to reason it out logically or compute properly the expense of financing a plant on a going basis, and the average estimator stops short when he gets to the end of his list of prices of visible things and wonders what he shall do next. It took a high Court ruling in the Kansas City Water-Works case, twenty years ago,* to compel the inclusion of the cost of reproducing the plant to an income-paying basis, and, ever since that decision, "going value" has been the bogy of valuators, utility commissions, and municipalities.

"Going value" is an unfortunate name. The Courts have attached those words to that part of the reproduction method which has to do with reproducing the financing of the physical property on a paying basis. As commonly used, it is nothing more or less than a necessary part of the reproduction process; as a principle, it is entirely simple and easy to understand.†

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* National Water-Works Company *vs.* Kansas City, 62 Fed., 853; 10 C. C. A., 653; 27 L. R. A., 827; 27 U. S. App., 165; July 2d, 1894.

† See "The Going Value of Water-Works," by Leonard Metcalf and John W. Alvord, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 326 (1911); also "Notes on Going Value and Methods for its Computation," by John W. Alvord, *Proceedings*, Am. W. W. Assoc., 1909, p. 184.

As reproduction is a cost method, and as cost is not necessarily value, the term "going value", generally used to designate the completion of the cost-of-reproduction method, is a misnomer. It would be better to modify the term to "going cost", when speaking of that part of the reproduction which it describes.

When we have completed our reproduction-cost estimates, including going cost, and when we have reasoned from this line of evidence and other lines of evidence to the value of the property, then we might properly use the term "going value" to designate the entire property value as a going concern, for then we are talking about value and not about cost.

A great deal of perplexity in valuation work and valuation discussion arises from just such confusion of terms as is here described, and this perplexity will continue until engineers get more clearly defined notions of valuation work, for it appears to be necessary to use brief descriptive terms in such discussions.

Reproduction A Cost Method.

A great many estimators, and most of the utility commissions, use the reproduction method as though it were a sort of a formula for the final determination of value. In other words, they unconsciously assume that they are completing their valuation, step by step, with each item of the reproduction, and get afraid during the process that the final result will not be exactly what they want; that is, they fear it will lead them somewhere where their reason would not fully approve. All this would be obviated if the estimator had clearly in mind the simple proposition that reproduction is a cost method, and, as such a method, when completed it may or may not be the real value. When one has completed the reproduction estimates, he always has the right to reason whether it furnishes him with a good approximation of the value. This is why the Courts keep reiterating that other things should be taken into consideration, such as past cost, for instance, where it is available, market value where it can be found, and, more particularly, the surrounding conditions, such as the need for the service, the value of the service to the consumer, the adequacy of the service, and such other factors as have been heretofore mentioned. All these surrounding conditions, with the proper analysis, will show whether or not the reproduction cost method can be safely used, and

if not directly used, what modifying influences would properly be taken into consideration.

If, for instance, a plant has recently been built in a town the chief industries of which have unexpectedly come to an end so that the town will inevitably depopulate, then the cost of reproducing such a plant is obviously not a proper measure of its value; or, if a new and better method of service be suddenly discovered or is soon likely to be discovered through new invention or change in the arts, then a similar result will ensue. On the other hand, if a plant is properly built, with reasonable skill, in a growing city, and has perhaps the most desirable kind of service, and is without engineering or financial errors, or dishonesty of management, or bad judgment in its financial development, then the value of that property may be even greater than would be indicated by the reproduction-cost method.

This matter of disassociating value from cost is one of the greatest difficulties that the engineering student of appraisal work encounters. It is so common an experience to most engineers to think of value as synonymous with cost, that the possibility of these two things being at times unequal is only received by the average mind with something akin to a wrench.

When an appraiser gets clearly in mind what reproduction is, that it is a cost method with many limitations, and that, although it is usually only one of the lines of evidence which should be taken into consideration, all difficulties with the logical details of reproduction will naturally and easily be resolved by the application of a little careful reasoning. It will be obvious that any doubts, for instance, about including pipe under pavements, interest during construction, ownership of services, overhead charges, preliminary expense, and going cost, arise mainly from the fact that the valuer has no clear idea of what he is doing or how it is to be useful when done. The valuer should never be afraid of a logical reproduction, but he should fearlessly and conscientiously follow it through to the logical end, not mixing it at any point with past cost, or deviating from the natural requirements of a practical and actual reconstruction in any way. When reproduction is thus carried out, the valuer should set it aside as one of the most important lines of evidence, and finally review it in connection with other lines of evidence and all the surrounding circumstances, many of which, though affecting value, cannot well be expressed in figures

at all, or can only be expressed partly in figures. His final conclusion as to what is the fair, just, and equitable value of the property in question, all things considered, should be reasoned out in the broadest kind of a way.

Summary of Reproduction.

In reproducing a plant and property, therefore, we may summarize the following principles of procedure as in accordance both with the opinions of the Courts and as generally agreed to by those appraisal engineers of long experience in valuation work:

First.—The reproduction should be along lines that are humanly possible, and in accordance with practical experience.

Second.—It is entirely a hypothetical and conceptual process from beginning to end, and, therefore, to be practically useful, must be founded on a long and wide practical experience.

Third.—It is to be based on a practical working plan, in which the time adopted for the proper procedure is one of the most important elements.

Fourth.—A hypothetical reproduction, to be practical, must be predicated on a reasonable construction and development period in the near future.

Fifth.—The costs for this near future must be based on the prices of the recent past, just as any engineer entering into any new work determines the probable near future costs from past experience.

Sixth.—The reproduction is not complete until the plant is financed on a paying basis, *i. e.*, earning a fair return.

Seventh.—To use this reproduction as a measure of the present plant, the depreciation of the existing property must be computed and allowed for.

Eighth.—Reproduction is a cost method, and, as such, may or may not be of value. When reproduction is completed up to a paying basis for the property, then, and not until then, have we a right to reason whether this cost method is a good measure of value.

Ninth.—If a plant is carefully and honestly built, if it serves a need in the community in the best possible manner, and if there is no better or less expensive service, then the cost of reproduction must of necessity be the least possible value that can be put on the utility, but the real value may often be in that case more than the reproduction cost.

Tenth.—On the other hand, if a plant has been recklessly overbuilt, or if the need for its service has diminished, or changes in the arts require its early abandonment, then the cost of reproduction will largely exceed the true value, unless materially reduced by an adequate allowance for depreciation of a general nature.

Past Cost.

It is interesting to note the tendency of the newer public utility commissions to emphasize past cost data as of major importance, and substitute it in whole or in part for reproduction cost wherever possible. Almost all appraisers have been through this experience. Later and more mature consideration generally converts them to giving more weight to the reproduction process. This tendency to emphasize past cost is not in accord with the opinions and reasoning of the higher Courts, which are constantly pointing out that it is the value of the property now that must be sought, and, though the Courts do not deny that past cost is a proper and suitable line of inquiry, they very properly insist that it is only useful because it may throw some light on the value of the property now. This line of thought of the Courts may be summarized, as follows:

First.—Past cost often does not include all the actual cash cost incurred in promoting the enterprise, nor does it usually show the deficits which have occurred below a proper revenue from time to time.

Second.—It lacks any indication as to how the property has appreciated, by reason of its own growth and, as well, the growth of the community in which it is located, or the growth of demand for its product from a given community.

Third.—It usually contains the cost of all obsolete structures, and, as found on the books, has usually no allowance for depreciation.

Fourth.—There is nothing to show the hazards in the original installation. Plants were often originally built when enterprises of a similar nature were new and untried, when the community was small, and the value of public service was unknown and unappreciated.

Fifth.—Markets for special classes of supplies, machinery, and labor, have fluctuated widely, and in some cases radically, so that any attempt to reason present cost from past cost must inevitably be a task of great speculative difficulty.

Sixth.—Granted that we could properly compute all the depreciations and the appreciations, and could determine correctly the proper deficits in revenue and make proper allowances for changes in the markets for material and labor, and compute correctly the hazards and cost of money under the original circumstances, we would not even then have a measure of value, but only a measure of cost, which might correspond with reproduction cost now, but this might or might not be the value of the property, as the case might be.

It is seldom possible to find an accurate and complete record of past cost, intelligently kept on a proper accounting basis, with proper separation of operating from capital account.

When the past cost of a public utility company is available and can be found from the books, it usually appeals strongly to the buyer side of the question, for the simple reason that it is often less in amount than the present value of the property, and usually less than the actual cost and the present cost by reproduction methods. Cases arise, however, where past cost is much greater than present value or replacement cost.

Undoubtedly, Courts, economists, and engineers would all like to have as definite and satisfactory a method on which to reason value as past cost would seem to be on the face of it, but, unfortunately, its face appearances are most misleading, for if we must find the value of the property now, that value is probably less controlled by past cost than any of the other cost methods from which the value can be reasoned.

The procedure adopted by some utility commissions, of utilizing reproduction methods for the structure cost, and adding to this amount past deficits in operation to produce a total, has always seemed to the writer to be singularly devoid of any rational basis of explanation, for it mutilates the reproduction of a property so as to render it illogical and incorrect. Some comments on this procedure may be summarized, as follows:

First.—It appears to be the result of a timidity about the reproduction method, which it is believed would not exist if it were fully understood that reproduction cost is not an arbitrary measure of value.

Second.—It leaves entirely uncompleted a valuable line of inquiry, that of reproduction logically carried out to its logical completion.

Third.—It does not encourage the summing up of the real present reproduction cost and the actual past cost as two entirely distinct and valuable lines of evidence, each to be used as a basis for further investigation and reasoning.

Fourth.—It appears to have resulted from an over-anxiety to reason about value before all the cost evidence is in hand.

Fifth.—It tends to discourage the adequate consideration of important factors which influence value, and which cannot always be well expressed in figures, and this effect is caused by reasoning on value at too early a stage in the proceedings.

Sixth.—It would seem that the consideration of past deficits is given too much prominence, because their inclusion in present value puts a premium on poor judgment or misfortunes in the past. The best managed and most valuable properties, under this method, would show the least value, and the most unfortunate and worthless would show up the best.

Seventh.—It would appear that past deficits belong to the past cost line of evidence, and are properly considered here, and not in reproduction cost evidence.

Eighth.—It is quite impossible to see how deficits of past operating cost have anything to do with the cost of financing a physical property into a paying income now.

Market Value.

Although the Courts have said that market value is one of the elements to be considered in valuing a utility property, as a matter of practical application, it is found that a broad and representative market for the securities of a utility company is not usually found, except in the case of the railways and the larger utilities in our great cities. In most of the smaller utilities a general market either does not exist for the property as a whole, or is so restricted as to be an unsafe guide as to its value.

In the process of reproduction, however, we apply the principles of market value indirectly. We may not find that the value of the property, as a whole, may be determined in this way, but by the process of an imaginary re-creation we may find the market value of materials and labor, machines and appliances, and so utilize market

value indirectly for at least a portion of the more tangible parts of the property.

In the larger utilities and the railways, where stocks and bonds are regularly listed and broadly dealt in, market value has more meaning, and should be reviewed, bearing in mind, however, its fluctuating character and its attempted prophetic indication of future rather than present values.

The market value of to-day is almost always built up on the hopes or fears for the morrow, and thus, to a large extent, may consist of too much optimism or pessimism for real present usefulness.

Market values, as indirectly introduced in the reproduction method, are also productive of embarrassment, for it is certain that the value of utility property, as a whole, does not fluctuate with the variations of the material and labor market on the detailed items of which it is composed, and this should always be remembered when the final reasoning is in progress.

On Reasoning From Cost to Value.

By far the most important part of the matter of determining a just and reasonable value of a property lies in summing up all the evidence obtained from cost or other methods, and reasoning from these facts and estimates to the broader question of value of the property, as a whole.

Singularly enough, this important phase of valuation work has not been generally appreciated by engineers, but it has been constantly emphasized or discussed by the Courts. Most engineers assume that the process of valuing is finished when the cost evidence is complete, and by that time they usually have become so beclouded by the mass of detail through which they have struggled that they are often mentally unfitted to undertake the broader and more important responsibility of reasoning to the final result.

It is true that if engineers are seriously to value a property they must familiarize themselves thoroughly with the details of that property, by carefully working out such lines of cost evidence as are necessary, but it by no means follows that they must neglect the broader aspect of the question when this preliminary and detailed work is done.

The general summation of value, from all the surrounding facts, is the part of the problem where the mental methods of the broad-minded business man and financier come into play, and it is precisely here that the business man becomes impatient with the Court and the engineers. Large buyers of public utility properties purchase such properties without any preliminary replacement estimates whatever. What a business man generally wants to know is the character of the city, the needs of the population, its enterprise and probable future growth, the presence or absence of business and manufacturing industries, and, as well, any probable competition or probable changes in demand. He wants to know the gross and net revenues actually existing. He will look carefully at the possibilities of economies, and especially at the probability of increasing the demand for the product or service. From these lines of information, and possibly comparative per capita costs of similar enterprises elsewhere, he will rapidly frame up in his mind a maximum and minimum value within which he can afford to trade for the plant, and the point at which the willing seller meets the willing buyer finally fixes the value of the plant in terms of money. Now, what appraisal work lacks to date is more of this kind of final review, well reasoned out after all the cost lines of evidence are in and complete.

To a man of affairs, accustomed to this kind of rapid valuation, the methods of the Courts, the appraisers, and the utility commissions look quite absurd—how absurd may be well indicated here by a rude illustration:

Appraising A Horse.

When an ordinary farmer or stockman wants to purchase a horse, he runs over in his mind the prices at which, ordinarily, horses have been selling, the uses and needs he has for this particular horse, the earnings the horse can make for him in the uses he will put it to, also the kind of a horse he feels will suit the conditions, and then he fixes, in his mind, the maximum amount he can afford to pay for such a horse, based on its usefulness, and looks for a willing seller having about the kind of a horse he wants.

Now, let us suppose the earnings of horses were regulated by law to only a fair return on their value. It would become necessary, under such circumstances, to seek a process which would show us what a

horse would be worth that could fill a particular need, at a regulated rate of earnings; the rate, in turn, being predicated on the value of the horse. This would reduce our stock farmer to a condition of mental coma, as it would introduce economic problems that are outside the domain of his practical experience, and he would probably have to seek the advice of a specialist.

Let us suppose that one of our modern, up-to-date utility commissions, with its engineering staff, were called in to help out, and suppose that it should proceed as we proceed to-day in the valuation of private property devoted to public use. First of all, it would probably take off the horse's shoes and weigh them, and compute the market cost of replacing them; then it would calculate the quantity of glue in the hoofs; get the measurement of the hide, and compute the fertilizer, estimate the grease, and value all the by-products at prices at which they could be purchased in the open market, and after all of this was done, it would total up the cost of reproducing the "physical property," so-called, and then perhaps suddenly discover that it had now only arrived at the cost of replacing a dead horse.

The problem would then be to find the added cost of producing a live horse, over and above the cost of a dead horse, and here the commission is in difficulty. The suggestion would occur to it, of course, that the hypothetical dead horse which it had just estimated, might be compared with the live horse which it is valuing, and the difference in cost might be called "going value," but this would not look very sensible. First of all, it would have to assume the cost of a live horse, which is assuming one of the results that are being sought.

Another method, following along the lines of the reproduction method, would be to give the carcass oxygen treatment, and, if possible, animate it into a live horse, which, after forced feeding and stimulants for a few days, might be able to do some work and earn certain revenue. The cost of all this biological proceeding might be compared, in the meantime, with the earning power of the live horse so as to arrive at "going cost," and be added to the carcass estimate. Some of the commissions would prefer to go back and see what it would cost to raise a horse from a colt, keeping track of the cost of feed, pasture, and care, crediting the horse with the work performed, and thus arrive at the net cost of the full-grown horse. Others would add the early deficits in earnings of the colt to the carcass value of the present dead horse, for

which they have completed estimates, and arrive at the net cost of a live horse in that way. Most engineers, new to appraisal work, would jump at the difference in cost between the dead and the live animal, but try to conceal the fact that they have guessed it under a mass of verbiage, which, when analyzed, is found to be evasive and meaningless. Once having arrived, by hook or crook, at the cost of a breathing working horse, most of our appraisal authorities stop, quite exhausted with such strenuous endeavor in these new fields of inquiry.

Now the difficulty of finding the going cost of a live horse, over and above the carcass cost, may seem far-fetched and not properly comparable with other kinds of property, but is it so far-fetched after all? What is it that goes into an incongruous mass of stone, brick, iron, and machinery that makes it a harmonious, useful, operating whole? Is it not brains, thought, patience, perseverance, foresight, ability, human energy, confidence, and wisdom? And these are but attributes of life; the characteristics of live things over and above dead things. When we make a property live, therefore, for useful purpose, we put something of the Divine human into it, which is little short of miraculous, and the operation of measuring this in terms of cost is necessarily a difficult and delicate task to which we must give careful thought, not only to the physical task of bringing a property to life, but to the psychological and ethical task of making it live usefully.

The stockman, however, might properly want to know what kind of a horse it was, after the appraiser had got it alive. Should it be classified as a race-horse, a carriage-horse, a cart-horse, or only just a jackass—and it might make a good deal of difference in earning power, as well as value, whether it were the first or the last?

Courts and utility commissions do not always disclose their mental processes, and, so far, we find on the records but little reasoning along the broader business lines which affect and surround the question as to whether the given utility is a race-horse or a jackass. Engineers, especially, are quite content if they can only get at the cost of a breathing, living horse, but it is pretty evident that this is an important question, and should be given a good deal more consideration in the future than it has had in the past.

Under the restricted methods of procedure, in which purely cost methods have such predominance, some utility properties have undoubtedly been undervalued, others must have been overvalued, and

unless we can come to appreciate the fact that surrounding conditions, good judgment, and skillful engineering tell in the value of a property, we shall not get away from this danger.

Now, the Courts and commissions being charged with the responsibility of determining values, which, in part at least, are dependent on rates, and at the same time of establishing rates which are a fair return on the value of the property, are under the most insidious temptation to lessen property values without appearing to do so, and without even intending to do so. This is due, in large part, to the unfamiliarity of the Courts and commissions with the business considerations which control the investment of capital and its expectation of profit, and also to the pressure of the public clamor as the buyer of service. Where appraisal authorities have yielded to these temptations, and particularly where they have failed to give due consideration to all the surrounding conditions outside of the cost evidence which go to make up value, they have, in some cases at least, placed the racehorse on a level with the jackass, and perhaps at times elevated the jackass to a dignity to which it was not entitled.

As has been said before, where a property is serving a real need in a community that is growing normally, and where its installation and management are without serious errors of judgment, the value of the property cannot be less than the cost of reproducing it in a manner that is humanly possible, less its depreciation. Ordinarily, it is worth more, and usually it would be considered poor financial judgment to sell a well-designed and well-established property in a growing town for the bare cost to reproduce it, were it not for the power of the Courts and the commissions to restrict value through the establishment of rates.

This brings us to the consideration of the economic law that when value depends on rates, rates must be such as to attract capital into the utility field or there will be no value commensurate with the necessary service; yet it is to be doubted if any actual rates so far established are of themselves high enough to account for the fact that capital is apparently continuing to move into certain kinds of utilities, especially into some of the municipal utilities. What are in reality the main inviting causes here may be summarized, as follows:

First.—The belief that the fundamental law of this country will protect property devoted to public use from confiscation.

Second.—The belief that legal supervision by non-partisan commissions will decrease the annoyance such investments have been put to in the past, and make the capital involved more secure.

Third.—The demonstrated fact that earnings from public utility investments do not fluctuate so greatly in financial depressions as do some other forms of investment.

Fourth.—The fact that the demand for the service will usually grow or can be encouraged to grow at a faster rate than the increase in population.

Fifth.—The probability, or at least the hope, that the value of the property in a public utility will grow with the community, irrespective of the actual investment in money.

Sixth.—The obvious intention of the Courts and commissions to treat such property fairly in the matter of reasonable rates.

So long as capital finds these conditions satisfactory, it will flow into the utility field in a selective way, but should these fundamental conditions be seriously altered or much changed for the worse, capital will grow timid, and will gradually withdraw.

In the case of privately owned municipal water supplies, for instance, the consumer public has, in the past, greatly overestimated the profits and underestimated the necessary value of the property required in the service, and, as a result, has so harassed such utilities that capital for many years has not only declined to enter the field, but has sought every opportunity to withdraw from it, even at a sacrifice. A further important instance illustrating this timidity of capital is noted at present in the steam railway field, where the adequacy of the present rates has come of late to be seriously questioned, and, as a result, capital has been less willing to enter the field during the last few years, with consequent higher cost to the railroads for money.

In reasoning from cost to value in public utilities, it is desirable to recognize that capital has properly treated the public, not only by good service, but by that further attention which wins and retains their good will, not that this of itself is an element of value, but it undoubtedly confirms and strengthens other elements of value.

On the other hand, we must take into consideration the fundamental treatment of capital by the public. If this is to be hostile and without intelligence, then values of all kinds will be lessened, and the

service will be ultimately destroyed. If it is to be scientific and just, then values will be maintained, and the service can be made efficient.

Now, the public is vitally and directly interested in the maintenance of values, just as capital is interested in having the public's good will, for without values we cannot have the use of capital, and without the use of capital we cannot have the reasonable service at fair rates, and we need and want the service.

In reasoning from cost and other evidence to value, however, there must be found some way to reward sound judgment, skillful engineering, and economical management, producing good service. This, none of the present utility laws as interpreted by the commissions, does; in fact, the present utility scheme of regulation, as now administered, distinctly puts a premium on poor or indifferent management, and takes away the incentive for economy. The next great problem in utility regulation will be to find ways of overcoming this defect.

In the meantime, there is no simpler way of recognizing good investments and careful management, where they exist, than by recognizing their effect in increased plant values, and this should be a part of the final reasoning in every valuation, just as much as indifferent judgment, absence of good engineering, and lack of economy should be similarly dealt with in the final summation.

It will always be very helpful if the maximum and minimum limitations to value be one of the matters to be reviewed before finally determining value, and, for this purpose, it is desirable, in the interests of the public, to reason out what would be the results of natural and economic conditions that would exist under a free competition that it is not wise actually to permit, and is not, in fact, intended to be permitted under our utility laws as they are now framed.

The average buyer in the open market always has his maximum price in mind, just as the seller has his minimum price, all of which is helpful in arriving at an exchange value. Helpful reasoning can be introduced along these lines in the final summation of public utility values. For instance, it would not be wise, on the one hand, to reward good judgment and skillful engineering in a property by fixing a value well above that which in an open and competitive condition would excite (if it were possible) the increased competition of a similar kind of supply or service, or the construction of a rival or duplicate property; nor, on the other hand, could we penalize dishonest financiering

and thoughtless construction by reducing values below the point where any service would be ultimately impossible, and the public finally be made to suffer.

Reasoning from the cost and other evidence to value, therefore, is a financial as well as a judicial and ethical problem, and if we would here do justice to the subject we must imitate the methods and habits of thought of the honest but alert business mind. When this economic study is followed through, it is striking to note how identical at every point are the common interests of capital and the public interest.

Conclusion.

There is probably no problem presented to the American public at the present time which is so difficult and complex as that of valuing the public utilities of this country. The value of these utilities is constantly fluctuating. It is dependent on many complex surroundings, and involves many difficult and delicate questions. To determine lines of evidence which will show the value of these properties in a way which the higher Courts have indicated, often involves the engineer, the economist, and the Court, in a complex hypothetical proceeding, which is only rescued from being absurd by being applied by competent and fair-minded men, with long practical experience behind them. The foundation for all this work undoubtedly rests in the law and its interpretation by the Courts, and the superstructure only is the field of the engineer and economist. Let us, therefore, as prudent men and experienced engineers be careful that we do not foolishly build on false premises and produce results which are "confusion worse confounded."

Once having founded ourselves properly as to principles, but one other great problem remains, that of holding to the judicial attitude. An entire paper might profitably be written on this subject alone. The engineer in appraisal work, to be successful, must keep himself unbiased and unprejudiced. At times he must resist the popular clamor for low, unjust, and unfair valuations, and at times he must resist the temptation to become the willing tool of powerful interests.

The preliminary bias in approaching public utility valuation has a great deal to do with progress in solving its difficult questions. Roughly, there are six classes of minds that ordinarily approach the problem of public utility valuation:

First.—Minds which are only familiar, by sentimental interest, with the rights of the public, and care little or nothing about learning about the conservation of property.

Second.—Minds which are fully familiar with the rights of the public, but are also open-minded and desire to learn about the conservation of property.

Third.—Minds which are, by training and environment, only familiar with the rights of property, and are not open-minded about learning anything about the rights of the public.

Fourth.—Minds which, by training and environment, are only familiar with the rights of property, but would also like to be unbiased and know more about the rights of the public.

Fifth.—Unbiased minds which are interested in just relations between the public and private property interests, but have not fully analyzed both viewpoints or had experience with both situations.

Sixth.—Minds which are naturally unbiased and are interested in and have mastered the just relations between the public and private property interests.

Classes 1 to 3 are worse than useless as appraisers, and, as writers, serve only to obscure and befog the problem of the just relations between the public and private property. They, too, often produce heat when there should be light. They are usually hopeless, and should not be entrusted with judicial functions in utility valuations.

Classes 2 and 4 may or may not develop into Class 5 or 6, depending on their inherent fairness and opportunity to study the opposing side of the question. Usually, they do not get enough opportunity to study the reverse side of the question until their opinions are too firmly fixed to be altered.

Class 5 may develop into Class 6 if opportunities come for sufficient experience and study on both sides of the question. They have analytical minds and judicial natures in the making.

Class 6 can only come from an unbiased and analytical mind that has had ample experience in representing both sides of the question.

To produce good results it is necessary, in appraisal work, to have practical experience in considerable amount, not always representing one side, but alternately representing both conflicting interests, in

order to appreciate fully the varied points of view which can be brought to this difficult matter.

Cross examinations in court, conferences with able attorneys, familiarity with the views of Courts and commissions, arguments with opposing valuators, experience on various kinds of appraisals: these all develop the practical, fair, and unbiased appraiser.

Nothing is so illuminating as to find oneself demonstrating value from two opposing viewpoints in two consecutive appraisals. A few experiences of that kind give one confidence in one's own sense of justice.

There is one golden rule for the engineering appraiser in valuation work:

Always value a property so that you would be satisfied with your valuation were you appointed by the other side at interest, and so that you will be satisfied with it ten years hence, as well as satisfied with it next month, if you actually find yourself on the other side of the issue in some other case.

APPENDIX.

Legal Decisions.

If the opinions advanced in this paper are well founded, then engineers who propose to fit themselves for valuation work should by all means make a careful study of the opinions of the Courts. What is greatly needed is a book, compiling in full, without partiality, the decisions and opinions of the Courts in valuation cases, all of which should be accompanied by a topical index. Probably this will be produced and published at some early date, as the importance of the subject now urgently requires that it be done.

In the meantime, a list of the more important decisions, especially those of the Supreme Court, is appended to this paper for immediate reference. The list may be profitably extended, but it is believed that most of the leading cases are included:

Ames *vs.* Union Pacific Ry., 64 Fed., 177; Digest, 2, 13, 26, 34, 38; Abstract, 48 to 50.

City of Bristol *vs.* The Bristol Water-Works Co., 23 R. I., 278, 49; Atl., 974.

Brymer *vs.* Butler, Pa., Water Co., 179 Pa., 250.

The Brunswick and Topsham Water District (Me.) *vs.* The Maine Water Co., 99 Maine, 371.

Baker *vs.* Gertside, 86 Pa., 498.

Chicago Railroads *vs.* Day C. C., 35 Fed., 866.

Railroads *vs.* State of Minnesota, 134 U. S., 418-456.

The Cedar Rapids Water Co. *vs.* City of Cedar Rapids, 91 Northwestern, 1051.

The Cedar Rapids Gas Light Co. *vs.* City of Cedar Rapids, 120 N. W., 966.

Cotting *vs.* Kansas City Stock Yards Co., 183 U. S., 79.

Contra Costa Water Co. *vs.* City of Oakland, Cal., 113 Pac., 668.

The Consolidated Gas Co. *vs.* City of New York, 157 Fed., 849-855.

Covington Gas Co. *vs.* City of Covington, Ky., 22 Ky., L. R., 796.

The Des Moines Water Co. *vs.* City of Des Moines, 192 Fed., 193.

The Galena Water Co. *vs.* City of Galena, 87 Pac., 735.

The Gloucester Water Co. *vs.* City of Gloucester, Mass., 60 M. E., 977.

The City of Janesville, Wis., *vs.* Janesville Water Co., R. R. Comm., Vol. 6, 628.

The Kennebec Water District *vs.* Waterville Water Co., 97 Maine, 185.

The City of Knoxville, Tenn., *vs.* The Knoxville Water Co., 212 U. S., 1.

Little Falls Electric and Water Co., 102 Fed., 663.

Monongahela Navigation Co. *vs.* U. S., 148 U. S., 312.

Munn *vs.* Ill., 94 U. S., 113, 133.
National Water-Works Co. *vs.* Kansas City, 62 Fed., 853.
Newburyport Water Co. *vs.* City of Newburyport, Mass., 168 Mass., 541; 47 N. E., 533.
The Norwich Gas and Electric Co. *vs.* City of Norwich.
The City of Omaha *vs.* The Omaha Water Co., 30 Sup. Ct. Rep., 615.
Peck *vs.* Chicago R. R., 94 U. S., 164; 24 L. Ed., 97.
The Pioneer Tel. and Tel. Co. *vs.* Westenhaver.
Queensborough Gas and Elec. Co. *vs.* Pub. Ser. Com., N. Y.
Redlands Water Co. *vs.* City of Redlands, Cal., 121 Cal., 365.
Stanislaus County *vs.* City of San Joaquin and Kings River Co., 192 U. S., 201.
San Diego Water Co. *vs.* City of San Diego, 118 Cal., 556.
San Diego Land and Town Co. *vs.* Jasper, 110 Fed., 702.
San Diego Land and Town Co. *vs.* Jasper, 189 U. S., 439.
San Diego Land and Town Co. *vs.* National City, 174 U. S., 757.
Slocum *vs.* City of North Platte, Nebr., 192 Fed., 252.
Smyth *vs.* Ames, 169 U. S., 466.
Spring Valley Water Co. *vs.* The City of San Francisco, Cal., 124 Fed., 574.
Spring Valley Water Co. *vs.* The City of San Francisco, Cal., 192 Fed., 137.
Spring Valley Water Co. *vs.* Schlotte, 110 U. S., 347; Sup. Ct., 4851.
Trust Co. of America *vs.* City of Rhinelander, Wis., 182 Fed., 64.
C. H. Vanner *vs.* Urbana W. W. Co., 174 Fed., 348.
Washington Gas Light Co. *vs.* District of Columbia, 161 U. S., 316.
Willcox *vs.* Consolidated Gas, 212 U. S., 19-54.
Wilkesbarre *vs.* Spring Brook Water Co., 4 Lack., Legal News, 567, 380.
Western Union Tel. Co., *vs.* Myatt C. C., 98 Fed., 335-342, 354.
Bothwell *vs.* Consumers Co., 13 Idaho, 568, 9e Pac., 533; also 24 L. R. A. (U. S.), 485.
Minnesota Rate Cases, 230 U. S., 352.

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THE VALUATION OF PUBLIC UTILITY PROPERTY.

By J. H. Gandolfo, Assoc. M. Am. Soc. C. E.

To be Presented November 18th, 1914.

Synopsis.

The primary object of this paper is to show that, in any valuation or appraisal that is to be made for purposes of rate-making or security issues, the proper figure to be used is the actual cost to date of the work in question; provided, of course, that there has not been extravagance, fraud, or gross mismanagement in the origin or development of the enterprise. At the same time, it is also shown that, although this value is a basic figure, it is not necessarily the only and final one to be used for any and every purpose for which a valuation may be undertaken. Reproduction or replacement cost, cost of reproduction less depreciation, etc., all have their proper place, and should be used under the proper conditions.

In order to show this, it has been necessary to go back to the fundamental beginnings of business, as exemplified in the private business, and trace the development of the private corporation and the public utility corporation from this origin, at the same time showing the similarity and dissimilarity between these three classes of organizations. With this as a basis, it is shown how the same principles can and should be applied in the valuation of public utility property as would be used in that of a private individual.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

This has been done under the three headings: I.—The Private Business; II.—The Private Corporation, and III.—The Public Utility Corporation. The last is again subdivided, and different phases of the subject are more or less fully treated, applying the principles as deduced from the other two divisions.

In investigating the public utility, it is also shown briefly why this class of corporation should be subjected to regulation by the State, this regulation being a necessary result of the circumstances attendant on the development of these corporations from the private business and private corporation.

Inasmuch as the final test of a physical valuation is its ability to stand a review by the Courts of last resort, the decisions of such Courts are referred to in the following pages wherever it has been thought necessary to show the trend of judicial opinion; also, it has been found advisable to quote at times from some decisions, not only for purposes of clearness, but for purposes of criticism.

In conclusion, the writer states that the actual cost to date is a figure equable to the investor, the promoter, the manager, and the public at large. The real object of a valuation is to obtain a figure equable to all parties interested in the particular case under consideration, and every case must be decided on its own merits, and with due regard to the conditions and circumstances immediately surrounding it.

Introduction.

In any work of valuation or appraisal, no matter for what purpose, there are of necessity many perplexing questions that are bound to arise, and their solution will tax to the utmost the experience and resourcefulness of the investigators. On the other hand, there are also fundamental principles governing such work, which, if studied and analyzed, will serve to elucidate many matters which are subjects of controversy at the present time.

In order to arrive at such fundamental principles and obtain a firm basis on which to found axioms for the valuation of the property of public utility corporations, it is necessary to investigate the development of this class of business from its beginnings, and to study it from economic as well as from financial standpoints. To attempt to unravel successfully the so-called mysteries of valuation and ap-

praisal, without such an investigation and without comprehending such first principles, is an absolute impossibility.

One of the principal reasons for so much confusion in regard to certain phases of the valuation of public utility property is that all investigators or appraisers fail to understand clearly wherein a public utility corporation possesses the same attributes as a private business, and, on the other hand, wherein it differs entirely. Therefore, it is necessary to show where the three classes of concerns, (1) the Private Business; (2) the Private Corporation; and (3) the Public Utility Corporation, are similar, and wherein they are radically and entirely dissimilar. In order to do this, and to make matters perfectly clear, it will be necessary to set forth some elementary facts relating to business in general, which perhaps are well understood when attention is called to them, but are forgotten or ignored by discoursers on these subjects.

In the following pages, it must not be supposed that the writer attempts to follow out in full detail all the intricacies of promotion, preliminary investigation, organization, underwriting, financing, or the dozen and one other steps that go to make up the history of any large business enterprise carried to a successful completion. On the contrary, the attempt has been made to describe briefly and in a clear, concise manner the principles that underlie such an undertaking, in so far as they are applicable to the valuation of public utilities. No reference will be made to any company organized simply for the purpose of placing stocks, without the intention of carrying the enterprise to a successful conclusion. It is assumed for the purpose of this discussion that the company is properly organized, is well managed, and that the intention is to return dividends to the investors. It is further assumed that, as far as ordinary business is concerned, there is free and unrestricted competition, and that there is no artificial control by combinations or trusts. Monopoly control is not considered, except in connection with the public service corporation. Such combinations in private business introduce elements into the investigation that are not germane to the scope of this paper.

I.—The Private Business.

Let it be assumed that an individual wishes to engage in a private manufacturing business of some kind, or that he is already engaged

in such. Aside from the transactions incident to trade requirements, the only direct financial dealings that such a concern would have with even a portion of the general public, in which any favors are asked, is the possible necessity of borrowing money from banks. If banks loan money on the concern's notes, such loans are secured by the value of the plant, by the value of the raw or finished material on hand or in progress of manufacture, or by bills receivable, all of which are tangible assets. Other risk of direct loss is taken by those parties selling to the concern raw or finished material on time payments. The notes are soon paid back, the same process being continued over and over again as the necessity arises. Therefore, the risk taken by the general public is practically nil, as it is taken only by those directly interested in the banks from which funds are borrowed, and who, acting through their boards of directors, have taken the chance of the concern not being able to meet its maturing obligations through some unforeseen accident, such as a falling market, financial panic, or similar causes.

In addition, in any dealings with such a concern, a creditor is further secured by the fact that the owner is personally responsible and that all his personal assets are liable as security in case other means fail; in strong contrast to a stock company, in which each stockholder is only liable for the amount of his stock, no matter what mismanagement may have developed or what debts may have been contracted.

If the concern is a co-partnership, the same conditions as those described are applicable, only in this case the risk is apt to be more extended, as there are two or more partners, and, in addition to banks, personal friends of the various partners are more likely to become involved in a failure, thus indirectly leading to a broader dissemination of the results.

If such a concern wished to extend its plant, this would be accomplished in one of two ways: (*a*) the owner would do with less personal returns and thus accumulate a surplus to be used for additions and improvements; or (*b*) he would raise the necessary funds from his banks, with property or other tangible assets as security, or from his personal friends, these loans to be paid back from the profits of the additional business before any benefit would be expected by the owner.

As to the returns to be anticipated from the business, this is simply a question of the law of supply and demand, modified by the ignorance, apathy, patience, desire, or necessity of the community. If the service is not satisfactory, or the grade of goods is not up to such a standard as the purchaser thinks is essential, as there is free competition, he can go elsewhere and obtain what he thinks is satisfactory. If too high a return is demanded, then, on account of the prospects of large profits, still other manufacturers will be induced to enter the field, engender more competition, and so reduce prices. One exception to this would be where some patented or secret process was in use, in which case the only check on the returns to the owner would be the desire or necessity of the public for the commodity, and its ability to pay for it.

As to the physical value of such a business, the owner would naturally carry the actual cost of the plant to date on the credit side of his balance sheet, minus such percentages as he had charged off for depreciation and obsolescence. If a fund for depreciation and obsolescence had been actually placed in trust for the renewal of the plant, it would be a tangible asset of the concern. If not, the owner would have to expect to renew the plant out of his own capital when the time came, just as he had originally built it out of his own funds; or else go out of business at the end of some period with his capital theoretically all returned, plus the amounts that the plant had earned, over and above the depreciation charges, during the years of its operation. It is not to be expected that an owner of such a plant would, under any circumstances, carry its present-day or replacement cost on his books, as such a quantity would have no place on his balance sheet, and, furthermore, would be a constantly varying figure. Neither would such a figure enter into the calculation of profits. Such profits would be based on the actual cost to date, not on any replacement value or present-day value.

If the owner of this business wished to sell it, or if some other party wished to purchase such a plant, the owner would have a perfect right to ask a price such as it would now take to replace his plant, less depreciation, or to erect a new one to produce the same results, which price might be greater or less than the original cost of the original plant, with all its additions and improvements to date. In addition to this purely physical valuation founded on present-day

values, the owner would also ask an additional sum to cover the fact that the business is a "going concern", that is to say, that it is organized, that it has customers, that it has an outlet for its products, and that it possesses the good will of its patrons. What such a figure would be, it is impossible even to estimate. It would depend on such circumstances as (1) the owner's desire to sell; (2) the buyer's desire to own such a plant, modified to a large degree by his ability to build a competing plant if not able to purchase; (3) by the state of the market for the commodity produced, whether rising or falling; (4) by general financial conditions. It is impossible to mention all the many causes, personal, economic, and financial, that might influence such a sale value one way or another.

If, instead of trying to buy the original plant, another party should build a new one and start operations, this new manufacturer would have had to consider (1) the possibly higher cost of the new plant, due to higher material and labor costs; (2) the necessity of building up a new business; (3) possible lessened income due to more competition; (4) lessened income in the early period of development due to faulty organization; (5) necessity of marketing the product; (6) failure due to over-production, etc. All these and many other conditions and circumstances would have to be carefully weighed and investigated before the enterprise was started; but, would the owner of the original plant make any changes in his investment costs? Would he increase the book cost of his plant, because some one else had built a more expensive plant to accomplish the same result? He would simply rest secure in the knowledge that he possessed this advantage over his rival. The new plant, perhaps costing more, must compete with the handicap of higher investment costs, modified perhaps to a greater or lesser degree by more efficient machinery and arrangements than those possessed by the old plant.

If the owner of a private business wishes to retire, there is no restraint to prevent him from so doing. At any time and at a moment's notice, provided he is solvent and has no outstanding contracts that must be fulfilled, he can close his doors, say "I am done and finished", and no one can compel him to continue in business.

The private concern never asks any favors of the municipality or of the State, such as the right to use the streets for car tracks, string-

ing wires, laying pipes, or other uses, or for the building of a railroad or the development of a water power. Nor can it invoke the right of the law of eminent domain to further its own affairs. It is subject to the same laws, rules, and conditions as the entire body politic, and thus, having accepted no special favors from the State, the private concern is not, and cannot be, required to return any special favors to the State.

Such, in brief, is an outline of a simple private business. It must not be supposed, however, that all details are covered or that all incidents which might arise have been mentioned. Any business is susceptible of endless modifications and extensions, and is highly sensitive to all those changes and developments which are constantly taking place in the business world. The outline is given as an exposé of the essential principles of such an organization. Its very fundamental simplicity is to be kept in mind, in connection with the investigation of both the private corporation and the public service corporation.

II.—The Private Corporation.

With the advent of the great stock company, with its large capitalization, its stocks and bonds, and its hundreds and even thousands of stockholders, we come to a new era in the business world, and here begins that intimate relation between the promoters, owners, managers (or whatever other name may be applied to the controlling interests of such a concern), and the general public at large. This relationship passes through all stages of development until the condition is reached where the concern becomes a public utility corporation, managed for and in the best interests of the people at large, who may now have no direct interest in the securities of the company, and yet, at the same time, may control its destinies through commissions and State legislatures.

All private corporations may be grouped in two classes, as follows:

(*A*). The corporations generally known as "close corporations," in which all outstanding securities are held by a few individuals, generally close business associates; and

(*B*). The corporations which, on their organization, offer to the general public stock or bonds, or both, and ask the public to purchase them and become part owners in the enterprise.

In regard to those corporations coming under (*A*), there are three reasons for organizing such a company:

(1) To provide against the disruption of the organization on the death of any prominent member. When a death occurs among the owners of a private business, it generally means the closing and winding up of the firm's affairs. On the death of any one holding stock in a corporation, however, the business is continued as before, under the direction of the board of directors. The securities held by the deceased simply pass into other hands.

(2) To limit the liabilities and responsibilities of those who hold stock in the corporation. As previously stated, the owner of a private business is liable to the full extent of his property, but the holder of stock is liable only to the extent of the amount of stock held by him.

(3) To enable an individual who is a bankrupt, to continue or engage in business, under the name of "manager" or "proprietor," the stock of such a company being held by a few of the "manager's" friends.

The corporations that come under (*A*), as long as they remain in this class, are practically private concerns, and no special attention need be given to them. On the other hand, however, the line between (*A*) and (*B*) is not at all clearly defined, and a company that is classed under (*A*) may at any time come into the other class by offering its securities to the public.

The primary object of organizing a large private company, such as classed under (*B*), is for the purpose of obtaining funds with which to carry out an enterprise of such magnitude that the money cannot as a rule be raised among a few individuals. Such an organization also enables an individual to take an interest in the company, by the purchase of a small amount of securities, he knowing that the amount actually invested by him is the only sum he risks, and that all his assets are not liable. The small investor can thus enter into and obtain the benefits of a large business, whereas if limited to the co-partnership arrangement, he would be unable to do so. In addition, such a company makes it possible for a few small investors, by thus pooling their capital, to engage in a business which otherwise would be controlled only by men of great wealth and financial standing in the community.

After the work of preliminary investigation and organization has been completed, such a company begins its public career by applying to the State for a charter of incorporation, and sets forth in such charter in what ways it proposes to exercise its activities. The promoters, then, on its incorporation, ask the general public to become owners or partners in the enterprise by the purchase of its securities. As it is impossible for all such stockholders to take an every-day active management in the company, such management is vested in a board of direction, the members of which are elected by the stockholders and are answerable to both the stock and bondholders.

The actual cash outlay for the enterprise, including all preliminary work, construction costs, and working capital must be raised by the sale of securities. In order to obtain this capital it is at present the general custom to offer the bonds of the company at a discount, not so much for the purpose of increasing the rate of interest on the investment, which is what such discount really amounts to, but simply to play on the psychological element of the human mind, which wishes to believe that it is getting something for nothing. Where an entire issue of bonds is underwritten by a banking house, or a syndicate composed of several such houses, part of such discount may cover the brokerage fee for handling and placing the securities and guaranteeing the funds when actually needed by the corporation. For example, if an issue of bonds is taken by a syndicate at 90, the syndicate takes the responsibility of placing them among the general public at some figure in advance of this and making the difference as its fee for the work. Such underwriting is more apt to be done with the securities of a public utility than with those of a private company. In addition to this discount on the bonds, a percentage of such purchase price, in stock of the company, is often given to each buyer of the bonds as a bonus, a part or all of which the syndicate may retain as part of its fee.

The promoters and organizers of the enterprise, in order to be remunerated for their work, also expect to be paid with a large block of stock, or with part cash and part stock of the company. In addition, as these organizers as a rule also wish to keep control of the company, enough stock must be issued and held by them so that a majority vote of the said stock will remain in their hands. Frequently, it happens that if a few individuals subscribe to a large percentage

of the issue of bonds, these men will insist on keeping control of the company, and so much stock is issued to them to insure such control. It is thus seen that the average corporation necessarily starts out with a certain amount of over-capitalization, or "water" in its securities, which it is impossible to prevent under present-day laws and conditions. This necessary over-capitalization, therefore, consists of the following:

(1) Promoters' remuneration in stock and cash;
(2) Discount on bonds;
(3) Stock distributed with the bonds as a bonus;
(4) Stock issued to keep control of the company.

On account of the way in which the business of the company is managed (by the board of direction), and because the company is controlled as described by stock issued for the purpose, the small investor, or "minority stockholder" as he is generally called, has very little voice in the real affairs of the concern. The Courts have ruled time and time again, when matters have been taken before them by minority stockholders, that, as a general proposition, in the absence of fraud or special circumstances, it is a question of "the greatest good for the greatest number."

More important, however—and of most consequence to the community at large—is the fact that the security holders of a large corporation are able to go to the banks and borrow cash, giving as collateral these stocks and bonds. Even the concern itself is able to borrow money from the same source, on the stock and bonds that it may have in its treasury. Thus stockholders of the banks and depositors who really may wish to have nothing whatever to do with some particular corporation, are brought into the matter entirely without their knowledge or consent. The financial world is filled with cases wherein banks have loaned to their full capacity on securities on which, in times of financial stress, they have been unable to realize, and have thus been unable to meet the demands of their depositors, although the securities in themselves were good and the concerns behind them perfectly solvent.

The stock company is thus seen to vary greatly from the private business, particularly in the following essential principles:

(1) The real "owners," or those who have actually supplied the capital, may have practically nothing to say about the management of the company, and have no control over their own investment.

(2) The company may have outstanding more or less securities which, having been issued to cover what may be termed "intangible elements," have nothing behind them in the way of tangible assets. When such securities are sold by the original holders, and pass into the hands of the public, there is nothing to distinguish them from those that are supported by tangible assets.

(3) These intangible securities, as well as those supported by tangible assets, may be used, through the medium of the banks, to borrow money from the general public.

(4) Returns in the form of dividends must be secured on a larger amount than the actual capital invested in the actual tangible business, as all securities issued are expected to bring in returns to their owners.

Aside from these matters, and as far as the actual conduct of the business itself goes, the private corporation does not differ materially from the private business, except in details which are not material for this discussion.

It is for reasons such as those briefly outlined that the State exercises control over all private corporations, and justly subjects them to rules and regulations that will protect, not only the public at large, but also those directly interested in the company. Some States exercise such control with a firm, and at the same time just, regard for the rights of all concerned, and others do so in a careless and haphazard manner that leads to fraud and corruption of every description. However administered, such control by the State is an absolute necessity. In other words, the State is beginning that unfortunately necessary paternal supervision over private business, which, later, finds its greatest expression in the direct control of the public utility corporation by the State.

III.—The Public Utility Corporation.

1.—General.—To-day we use the terms, public utility corporation, public service corporation, or quasi-public corporation, without a very distinct idea as to just what is included or should be included by these terms in the way of corporations. With our rapid development, the purely private business of to-day may become the public

utility of to-morrow. Almost everything now classed as a public utility was unknown a short hundred years ago. In ancient and medieval times, great cities existed and flourished, there were no railroads, and steam was unknown. To-day, however, the railroads stand at the head of our public utilities, and how could a modern city exist without them? How long will it be, therefore, before other lines of business endeavor now considered purely private matters will be classed as "public utilities?" As time goes on, more and more concerns, with more and more diversity of products or lines of endeavor, are being brought into this category. Therefore, what is a public utility, and how can it be defined and distinguished?

A public utility corporation, or a public service corporation, may be defined as a private company which exercises its functions in a direction that is of vital importance to the members of the community reached by its activities. Webster says:

"Private corporations, including all others of a civil nature, as ordinary business corporations and those corporations such as railroad companies, lighting companies, water companies, etc., organized or chartered to follow a public calling or to render service more or less essential to the general public convenience or safety, and now often called public service corporations or sometimes quasi-public corporations."

The great distinguishing characteristic of the public utility corporation, which separates it from the private company or private business, is, that it is organized with the idea of working to a certain extent in direct co-operation with the community—and by community the writer means either a village, a city, a State, or a Nation. Such a corporation asks certain definite concessions and grants from the State, as a preliminary to starting in business at all. For example, the railroad asks the right to cross or close highways; to take property from citizens by right of eminent domain (and, of course, for a just compensation); to build bridges across navigable rivers; to narrow the waterway of smaller streams, or even to change their course and direction by means of embankments and fills; the street railway, the water company, the gas company, the electric light and power company, all ask the right to use the public streets to conduct their business; the power development company asks the right to dam rivers and use the water for private gain. Of transcendental im-

portance, however, both to the corporation itself and to the community at large, is that of necessity this class of corporation is a monopoly; if not *de jure,* at least *de facto.*

In return for all these special privileges granted to it by the State, the company promises necessary or useful service of some kind to the citizen. It is the abuse of such privileges on the part of the corporation and, on the other hand, onerous conditions imposed by States on such companies, that have brought about the necessity of just regulation and control of them by the State. That such regulation is equable, few will question at the present time; but how it is to be accomplished, and on what basis, is another matter.

As a basis for the control of any public utility, it is necessary to know the cost, present value, or fair value, of all the property, both tangible and intangible, that goes to make up the utility. Without such knowledge it is impossible to form any definite plans to regulate these companies.

It is in regard to such value that the first serious divergence of opinion occurs among appraisers, and this lack of unanimity exists, not only among engineers, but also among financiers and the judiciary. For example, in making a valuation, should actual cost to date, cost of reproduction new, or fair or market value, be used? Should such matters as depreciation, obsolescence, and inadequacy, be taken into consideration? Inasmuch as the rights and privileges of the company have been conferred directly by the State, should such intangible elements as going concern values or franchise values be considered?

In any valuation work that is to be subjected to judicial review (and practically every appraisal must sooner or later stand this test), it is necessary to give careful study and consideration to Court rulings on the subject, in order that the work may hold through such an ordeal. It must not be forgotten, however, that a Court is compelled to render a decision on the evidence as presented before it. If a case is presented in a different manner, supported by the proper evidence, the ruling of the Court is apt to be very different. The writer's experience in court has shown conclusively that an attorney, although he may have a good case, through lack of proper study and a thorough understanding of its principles, may present it in such shape as to leave the Court no alternative but to give an adverse decision in the matter. The entire subject of valuation and appraisal is still in its

infancy, and the records of the judiciary show that higher Courts have repeatedly reversed the decisions and rulings of lower ones. Therefore, too much dependence cannot be given to judicial precedent in matters relating to public utility valuation and control.

After having given careful study and thought to the matter, coupled with practical experience in this field and an investigation of commercial relations in general, it is the writer's opinion that actual cost to date is the logical and just figure to be used as a fundamental basis for public utility control. Although such actual cost should be used as a basis, it does not follow that this is the final figure that should be used for any and all purposes for which a valuation may be made. The writer does not agree with those who hold that the same ultimate figure should be used, no matter what the purpose of the valuation. The object of the valuation very properly should affect, not only the final figure, but possibly the method of conducting the examination as well.

2.—Valuation for Rates and Security Issues.—In any valuation for the purpose of rate-making, or for regulating the issuance of securities, actual cost to date is the final figure that should be used, provided there has not been fraud, incompetency, extravagance, or anything of a kindred nature in the development of the utility. Such cost should include:

(1) All preliminary work of developing and organizing the enterprise, including promoters' fees.

(2) Preliminary investigations and preliminary engineering work of all kinds.

(3) All final engineering work, including the preparation of final plans, supervision of construction work, etc.

(4) All construction work of whatever nature, up to the starting of the enterprise.

(5) All other overhead charges during construction, which include administration, organization, legal expenses, taxes, insurance, interest, etc.

(6) All additional work of new construction built since operation started.

(7) Where new work has replaced old, and the cost of this new work is greater than that of the old, the difference in cost between the old and the new must be included; but care must

be taken that maintenance is not by any chance classed as new work.

(8) All working capital.

The actual cost should not include:

(9) Anything in the way of excessive promoters' fees or profits, or excessive salaries during construction.

(10) Anything in the way of engineering or construction work that partakes of the nature of fraud, incompetency, or extravagance.

(11) Discount on bonds.

(12) So-called development expenses during the early stages of the operation of the enterprise.

(13) Any part of the development that is not actually used or useful in the operation of the utility for the public benefit.

(14) If new work has replaced old, such as a heavier bridge, or a new cut-off, and the cost of this new work is included in the valuation, under no circumstances must the cost of the now abandoned and useless work be included also.

Many of the most important legal decisions relating to valuation uphold the present or fair value, as the equable one to use for either of the above-mentioned purposes. In view of the general increase in construction costs, this practically amounts to cost of reproduction less depreciation. (See Ames *v.* U. P. Ry. Co., 64 Fed., 165; Smyth *v.* Ames, 169 U. S., 466; Willcox *et al. v.* Consolidated Gas Company, 212 U. S., 19; Cedar Rapids Gas Light Company *v.* Cedar Rapids, 223 U. S., 670.) The principal grounds on which these decisions are based, and on which the advocates of this system of physical valuation found their arguments, are as follows:

(1) Many of the older corporations, such as the railroads, were organized, and the utility built and operated, before State regulation in detail and public service commission control were thought of, and that, therefore, it is not now equable to deprive these companies of the increased value of the property.

(2) Also, in the case of these older utilities, it is almost impossible to get any idea of the actual cost of the property to date.

(3) The company is entitled to the benefit of any increase in the value of its property since construction, and if such increase is not

allowed, it amounts to confiscation of property within the meaning of the Fourteenth Amendment of the Constitution. Conversely, if the property has decreased in value, the company must stand this loss.

In the case of some of the older public utility companies, there are undoubtedly grounds for the contention that not to allow the increase in value which has occurred would perhaps amount to confiscation. On the other hand, such companies have had a long period of unregulated existence in which returns may have been received sufficiently in excess of what are now considered equable to the community to cover this matter. In order to reduce physical valuation to some standard basis, it may be necessary to ignore some such claims in the future. No change, however slight, can be made in our commercial and financial systems without working some injustice. In a change so potent as that of the regulation of these companies, it would be strange indeed if some unavoidable injustice was not done.

It is also to be remembered that all these old corporations received their rights, immunities, and special privileges directly from the State. It is a fundamental principle of the old English law, that no legislature can limit or restrict the actions of any succeeding legislature. Therefore, any such body can alter or modify the acts of any previous similar body, and can even revoke franchises or other special privileges. Such acts, however, on the part of a legislative body, are in their turn modified and controlled by decisions of the Supreme Court (which have always upheld the Fourteenth Amendment) that property cannot be confiscated, directly or indirectly, but can only be taken by due process of law. (See Stone and others *v.* Farmers' Loan and Trust Company, 116 U. S., 307.)

It must likewise be kept constantly in mind that the price of all material and of all labor entering into the construction of public utilities, has risen in value, with very few exceptions, since the older utilities were built, and, therefore, in spite of improved methods of construction, the present-day value is higher than the original cost. In the case of land, this has often increased in value three or fourfold, and sometimes, especially in the case of city property held by railroads, eight or tenfold. It is for this reason that all public utility companies wish the present-day value to be used as a basis for rate regulation or for the issuance of securities.

Although many decisions uphold present or "fair value" as the proper one to use for rate purposes, there are others that uphold actual cost to date as the just figure to be used. (See San Diego Water Company *v.* City of San Diego, 118 Cal., 556; 50 Pac., 633; Brymer *v.* Butler Water Company, 179 Pa., 231; 36 Atl., 249; Interstate Commerce Commission, in Advances in Rates, Western Case, 20 I. C. C. R., 307.)

In addition to these decisions, if some of those which uphold present and fair value are examined, places will be found which indicate that the Court is not at all sure of its stand. For example, in Smyth *v.* Ames, Justice Harlan said:

"And in order to ascertain that value, the original cost of construction, the amount expended in permanent improvements, the amount and market value of its bonds and stock, the present as compared with the original cost of construction * * * are all matters for consideration * * *."

In this decision the fact of original cost being an important element in an appraisal is admitted.

In Ames *v.* Union Pacific Railway Company, Justice Brewer said:

"Nevertheless, the amount of money that has gone into the railroad property—the actual investment, as expressed, theoretically, at least, by the amount of stocks and bonds—is not to be ignored, even though such sum is far in excess of the present value."

This opinion needs no comment.

In Willcox *v.* Consolidated Gas Company, Justice Peckham said:

"If the property, which legally enters into the consideration of the question of rates, has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule. We do not say there may not possibly be an exception to it, where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such increased value unjust to the public."

Justice Peckham here lays stress on what he terms "the general rule," but the question naturally arises, What is this general rule, who made it, and why should it be used as a basis of a decision by the Supreme Court of the United States? The Justice further says, that if the value has greatly increased, it might be unjust to the public to base rates on such greatly increased value; but, if this

would be an injustice to the public, why would not a rate based on any increase in value be unjust? Who is to decide in such a case where to draw the line between such increase on which rates would be just and where they would be unjust?

Another argument against the use of this increase in value is that a public utility corporation does not go into business with any idea of speculating in its investment, any more than a private manufacturing concern does. The increase in value has not been caused by any direct efforts on the part of the company, and amounts to an "unearned increment" on which the public should not be required to pay rates.

As to the impossibility of obtaining the actual cost of a utility, this contention is also doubtless true in the case of some of the older concerns, but the writer thinks that the difficulties are greatly over-estimated in many cases. In any event, this is no excuse for putting all other utilities in the same category. If the actual cost to date is available, or if it can be obtained with reasonable efforts, to ignore this value and appraise the property on any other basis, such as reproduction cost, is to enter unnecessarily into the realms of speculation and surmise, which is so severely condemned by Mr. Justice Hughes in the Minnesota Rate Cases, in referring to land values (Simpson *et al. v.* Shepard; Same *v.* Kennedy; Same *v.* Shillaber, 230 U. S., 352).

Some few years ago the writer was chief assistant in an important investigation and appraisal of a large private industrial plant, which work was used in a suit in the United States Circuit Court involving a claim of $30 000 000. In this case the original cost of the plant was available, but it was necessary to examine carefully the actual property, and check conditions as they existed with the actual cost of the work. In addition, it was also necessary to follow back commercial conditions for the nine years previous to the appraisal, and discover what results would have been accomplished under such conditions. Of course, this was a long and tedious process, and absolute accuracy was essential on account of the legal aspects of the case. There was no reason, however, why these conditions could not have been traced back twenty or thirty years as well as nine; and, if the actual cost of the plant had not been available, it would have been perfectly feasible to appraise it at any period in the past (within a

reasonable time), using such prices and values as applied to that period. In this case, the present or fair value of the plant would have had no possible significance.

The actual cost to date represents the actual amount of capital invested in the enterprise; it is the amount on which the investors expect returns, and, if the enterprise is a success, on which they should receive dividends. This figure is also a constant quantity, it only being necessary to correct it for additions made from time to time; whereas, the present value (or value obtained by any other method) is a constantly changing quantity, it being necessary to alter it as conditions and prices fluctuate.

In order to demonstrate one phase of this problem, let it be assumed that the cost of reproducing a utility would now be less than when the plant was placed in operation. If the State or the Court should say to the corporation, "You must reduce your capital under the conditions as they now exist," the writer fails to see why this is not confiscation of property within the meaning of the Fourteenth Amendment. It is not just thus to wipe out invested capital, as Justice Brewer pointed out in Ames *v.* U. P. Ry. Co., 64 Fed., 165.

One of the arguments against using the actual cost under such circumstances is the claim that there are many utilities, especially railroads, which could not charge a high enough rate to pay a return on the investment without working a palpable injustice to the public; and the familiar illustration given is that of a branch railroad built to tap a logging or mining district, where, after the natural resources are exhausted, there is not enough traffic to pay operating expenses, aside from interest on the investment. This is all true, but a problem of this kind should be treated in an entirely different manner. Such a road never is, and never would be, constructed in the same substantial manner as a permanent line built for all time. The entire line and equipment would be of a temporary nature. When such a road is contemplated, it is the promoter's business to take such facts as its temporary character into consideration, and a return must be obtained during the years of successful operation, such as will, at the end of its period of usefulness, have returned the capital to the owners, plus all expenses and a fair rate of interest on the invested capital; and under these circumstances such rate of return should be allowed by public service commissions. In other words,

the entire capital of the railroad must be amortized by the time its period of usefulness is over. If, however, a road has been built, and through unforeseen conditions has ceased to pay a return on the investment, then it becomes a simple question of an unsuccessful business enterprise, with bankruptcy, a receiver, and a winding up of the company's affairs. This is the way a private enterprise would be treated, and there is no reason why a public utility should not be handled in the same manner.

A further objection to this method of treatment may be raised, on the ground that the utility is of absolute necessity to the community, as, for example, a water company supplying water for household use and fire protection. Such an objection is well founded. However, a community cannot compel a company to operate indefinitely at a loss, so, under such circumstances, the community should operate the utility at its own expense until such time as a proper sale value can be placed on the property and the utility is acquired by the community, such value to be founded on "market value," or, in other words, the "bankrupt sale value," and not on either actual cost to date, replacement value new, or any other value.

In many appraisals it will be found that there are often items which, if a fictitious value such as present-day cost is given to them, will lead to embarrassing and often ridiculous results, whereas, if their actual cost is used there is no chance for an injustice, either to the company, to the rate-payer, or to the investor. The following are given as some typical examples:

According to an old print, the great Tring cutting, made on the line of the London and North Western Railway in 1837, was excavated by hand and the spoil was removed in hand-barrows guided by men and pulled up inclines on the sides of the cutting by ropes passing over pulleys and worked by horse-power. If such a piece of work was to be valued by cost of reproduction, should the same conditions be duplicated, only with present-day prices for labor and material, or should it be assumed that the most modern methods would be used, with steam shovels and construction trains? In either event, it would mean entering into the realm of speculation and surmise, when the facts are there before the investigator.

On many of the early railroads built in the United States, temporary trestlework was constructed of timber obtained along the line.

It surely would not be just to estimate a value for such works based on present-day figures for labor and material entering into them.

In developing a water power, it is common practice to obtain the timber for the coffer-dams, and also the sawed and dressed lumber for form work or other uses, from the forest immediately contiguous to the site of the development. The writer knows of cases where such sawed lumber has been supplied at from $10.00 to $12.50 per m. b. m. Later, when all this timber has been cleared off, it would not be equable to estimate such lumber at a price of from $25 to $30 per m. b. m. just because it would now cost the latter figures to obtain it at the site. On the other hand, the cement used in many of the masonry dams in the Western States was obtained at an exorbitant figure. As lines of transportation develop, and markets come nearer these sites of power development, there is no reason why the value of the dam should be placed at a lower figure, because cement can now be obtained cheaply.

Several years ago the cost of a certain class of steelwork, erected in place, was 4.07 cents per lb. Since that time, this price has not been reached, but the same class of work has been done as low as 2.87 cents per lb. (an exceptionally low figure). This is a difference of 29½ per cent. It would not be just to compel the railroad which had paid the higher price to reduce its capital to suit some arbitrary figure which would be a mean between these two extremes. The capital for this work was honestly invested, and no part of it should be wiped out.

In regard to those items mentioned in the first paragraph of Section 2, which should and should not be included as capital in a physical valuation (numbered from 1 to 14, inclusive), it is necessary to discuss some of these more in detail.

Referring to (1), "Promoters' fees", it can readily be explained why this item should be taken at actual cost. This fee depends on the circumstances immediately surrounding the specific case under consideration. What would be excessive under some circumstances, would not be commensurate in others. One of the largest hydro-electric developments in the world was advocated for more than 10 years by its promoters before final completion. Such devotion to an ideal is worthy of more substantial reward than if 6 months or 1 year only was spent in such work.

In some cases the promoter's fee may also include and cover all preliminary work of developing and organizing, in which case the one item only must be included in the valuation. For these reasons, if this item is estimated on a cost-of-reproduction basis, it amounts simply to a guess as to what is equable.

As to those items included in (5), "All overhead charges during construction," there is no way by which they can be estimated on a present-value method with any degree of accuracy. The system usually adopted is to calculate such items as a percentage of the inventory reproduction cost of the utility. Such a figure might be very much less or very much greater than the actual cost. The amount of each of these items depends on the particular enterprise, its period of construction, and the conditions immediately surrounding it. Therefore, they should be included at actual cost only.

In regard to interest on capital during construction, there is one element of this matter which the writer does not recall having heard mentioned by any author on this subject, and which tends to reduce the total interest payable before the completion of the enterprise. This is that final payments on contract work are sometimes not made until the utility is in operation. The writer has drawn up many contracts for construction work on public utilities in which the following clauses were inserted:

"The company shall, however, deduct and reserve ten per cent. from any periodical payments falling due for work done and materials furnished hereunder, to be paid only at the expiration of thirty days after the works shall have been completed, approved, and accepted by the engineer * * *."

"The works shall not be finally approved * * * until all machinery and apparatus furnished hereunder shall have been regularly operated in the manner for which the same are intended to be used for a period of 30 days from the date of starting the same in regular operation * * *."

Referring to Item (7), cases may occur where the new improvements possibly cost less than the original investment. Any cases such as these must be cared for by a fund to cover depreciation, obsolescence, and inadequacy. Referring to the private business, it was seen that such a concern usually charged off a certain percentage of the cost of the plant from year to year, until the capital invested was theoretically all returned. As a public utility corporation cannot do this,

and go out of business at any time at its own volition, the securities covering any such abandoned or superseded property must be retired by withdrawals from the fund set aside for that purpose, unless there are other tangible assets not covered by securities to take the place of the useless property.

Taking up those items which should not be included as capital, and referring to Item (9), it has often been the case in the past that the promoters of the enterprise received such a large remuneration that the company was strangled with overhead charges before it had a chance to start. Such excessive fees must not be included in any physical valuation, and only a fair amount must be allowed under such circumstances.

Thus any excessive salaries of officers or of those in charge of the construction of the utility must not be included. If there has been anything of this kind in the development of the enterprise, only such an amount must be included for this item as such first-class direction and supervision could have been obtained for in the open market.

"Discount on Bonds", Item (11), must not be charged to capital. This item, no matter in what light it is considered, amounts to an increased rate of interest on the capital actually invested. As interest is considered an operating expense, and a sum is set aside from gross receipts to take care of it, so discount (which is only deferred interest) should be taken care of by equal increments set aside from gross receipts from year to year. That is, discount must have been amortized by the time the bonds mature.

Even if the discount includes the brokerage charge, it is still interest that must be met, and so should be amortized as explained previously. The brokerage charge is payment for the use of money; therefore it is a true interest charge and should be treated as such. All commission rulings in regard to this item follow this method in dealing with the question.

Referring to Item (12), it will be noted that early development expenses are excluded. The reason for this is that, according to the writer's view, these are strictly operating expenses. If a utility is properly managed, the development stage never ceases. The training and education of employees for their various duties never end. The advertising department is always attempting to obtain more and

more patronage for the utility. As to errors of design and construction which may develop during the early days of the operation of a utility, and the correction of which so many appraisers claim should be paid for out of capital, these also should be paid for out of profits, if there is money to pay for the work, because there is practically no limit to which such changes can be carried. For example, on railroads expensive tunnels, long cut-offs, great trestles, deep fills, and heavy bridges, are to-day being constructed simply to correct such errors of design and construction; and if such corrections and betterments are to be capitalized as new development, what security is left for the obligations already issued against the original and now abandoned or superseded work? In other words, "development" should be the watchword of every utility, whether in economy of operation, increase in business, or increase in satisfaction of service. The appraiser who attempted to include any such items as early development expenses in a physical valuation, either on the basis of actual cost to date, or present value, would be confronted by a peculiar problem. He would not know where to begin or where to leave off. Even if detailed accounts of the company were open to him, he would be no better off. It would simply be a matter of his personal opinion as to what items of this class of operating expenses he called development expenses, and what he called operating expenses, and his personal opinion, also, would alone be his guide as to where in the corporation's history he would stop charging development expenses to capital and charge them all to operation.

Furthermore, relative to this matter, it is to be remembered that most utilities have been slow in growth, and that it has not been necessary to train forces or build up an organization from absolutely ignorant material to handle the complicated conditions of to-day. For example, in the railroad world, the transition from the small systems beginning in England in 1825 with a few miles of track, tiny cars, and 8-ton locomotives, to the great transcontinental lines of to-day, with thousands of miles of track, cars of 110 000-lb. capacity, and great Mallet compound locomotives, was not made in a single stride. It was a slow development, and the corporations gradually expanded and improved to meet the conditions as they arose.

In regard to Item (13), if a utility has been constructed on a larger scale than is necessary to supply the service properly, then the cost

of such excess size must not be included in the valuation. There is an exception to this, where a utility has made provision for future extension and development to a limited degree; but it must be only to a limited degree. If the utility has made such provision out of all proportion to the needs of the present community, then it cannot expect to obtain rates on the cost of the same. The capital must have been wisely and judiciously invested. (See Smyth *v.* Ames, 169 U. S., 466; Pioneer Telephone and Tel. Co. *v.* Westenhaver, 29 Okl.; 118 Pac., 354; San Diego Land and Town Co. *v.* National City, 174 U. S., 739.)

3.—Real Estate.—In regard to the valuation of real estate, a number of elements enter into this problem, which differ from those affecting the valuation of other tangible property, and, for this reason, this item is generally considered separately.

All public utilities have the privilege of resorting to the rights of the law of eminent domain to acquire property, if such property can be shown to be necessary for the proper development of the enterprise. Thus, an individual, although he may foresee that his property is going to enhance greatly in value in the future, can be forced to part with it on the basis of its value at the time condemnation proceedings are instituted.

Those who uphold the present-value theory of valuation maintain that land should be included at its present value, as determined by the price of similar property in the immediate neighborhood, plus a percentage to cover the expense of what it would now cost the utility to acquire such land. That it costs a utility much more to acquire real property than its actual market value, is now such a well-known fact as to need no argument here.

Such extra cost in the case of urban property consists of legal expenses incident to the transaction, plus the costs of condemnation proceedings, if final resort is made to the law of eminent domain. Also, more particularly in the case of a new utility, there may be the cost of damages to adjoining property, caused by the very fact of the utility being in operation at all. In the case of country property, there is also the cost and value of such elements as temporary and permanent severance damages, plottage, and continuity (the three latter items applying more particularly to railroads). As all such extra values and costs are known to the average seller, he asks such

a price, over and above the fair market value of the land, as he thinks the corporation will pay, without resorting to condemnation. As time is of very great importance, and condemnation is apt to be a slow process, the corporation is generally willing to pay even more than this would cost.

For city property, the percentage of increased cost to the utility runs from 120 to 300%, and, in the case of country property, from 200 to 800% of the fair market value of such property without improvements.

In discussing this question, the following facts are to be kept in mind: In the case of many utilities, especially the railroads (our leading public utilities), land was purchased, even in the cities, at a very low figure. In many cases, large grants of land were made to the railroads, sometimes by individuals and communities for right of way and station sites, and by the United States Government for similar purposes. In addition, in some cases, large tracts were also given by the Government simply as a bonus for building the railroad at all. All such grants were made on the understanding, if not specifically stated, at least implied, that the corporation was to return useful and satisfactory service in some form to the community.

Almost without exception, the value of such land, whether obtained by purchase or gift, has increased enormously, such increase having been caused in large measure by the very fact of the utility being there at all. All utilities now claim that they should be allowed to include in a physical valuation all such land, not only at its increased value, but also at a value obtained by increasing such present market value by the same proportion as the original value of the land was increased in cost to the utility. In other words, the utility corporation wishes to put all conditions back as they would be without the utility, except the greatly increased value of the land, then assume that the utility wishes to acquire such land, and must go through all the processes to do so, increasing the value of the land by the percentage as already determined, and this to apply whether the land in question was obtained by donation or purchase.

To make this perfectly clear, assume the following concrete case. Land at a value of $100 per acre was acquired by a utility corporation at a total cost of $250 per acre, or 2½ times its market value, and adjoining land of the same value was also donated to the company.

All such land is now worth $1 000 per acre. The contention of the corporations is that all such land, including that donated, should be included in a valuation at a price of $2 500 per acre.

Even if an appraisal is being made on a present-value basis, the writer fails to see any logic or justice in such a contention. If the land is to be included at its present value and such value is high enough to cover all the original cost of acquiring it by the corporation, then no multiplier whatever should be used. In other words, the utility corporation, in the case of land, wishes to assume a set of theoretical conditions that could not by any possibility exist, and then found value on such conditions. It is these very elements that are so severely criticized and condemned in the Minnesota Rate Cases (230 U. S., 352) and that were decided against the utility corporations.

As a further example, suppose a community has presented a station site to a railroad. The corporation claims the right to ask rates on the value of such property and to issue securities against it; but, as rates are supposed to be a return on capital invested, and as securities are supposed to be issued to obtain capital to build the road, how, by any possible process of logical reasoning, can donated property be used for such purposes?

How untenable this present-value theory of land is can be demonstrated by illustrating an extreme case. Assume that a railroad had acquired terminal facilities in a city years ago, when land in the immediate vicinity of such property was cheap and unimproved. Now, suppose that the neighborhood improves, that the city grows up to and surrounds the terminal, and that the adjoining property becomes improved with expensive hotels and office buildings (this assumption is a fact in many cities). It would surely not be equable to value such terminal land for rate or security issue purposes on the theoretical supposition that it was all, or any part of it, occupied with such buildings, and that the public utility corporation (the railroad) must now pay the full value of such land, with all the improvements thereon to date, in order to acquire it. On the other hand, if, in order to provide for necessary extensions, the corporation was compelled to purchase any such land with such improvements thereon, then, even though the improvements were destroyed, the land should be valued at the total cost to the company.

Therefore, land or real estate should be included in any physical valuation on the same basis as any other property.

4.—Depreciation, Obsolescence, and Inadequacy.—In discussing the matters of depreciation, obsolescence, and inadequacy, most appraisers seem to take the stand that some definite scheme must be decided on at the commencement of the operation of the utility, and adhered to without any change, by which its total value will be returned to the owners at the end of some fixed period, which is as near as may be the end of the useful life of the plant. The following theories have been developed to produce this result:

(1) Replacement method for depreciation;
(2) Sinking-fund method for depreciation;
(3) Straight-line method for depreciation;
(4) Equal-annual-payment method for depreciation.

It is not necessary to discuss these methods separately. The trouble with all of them is, that, in order to work them out and discuss them at all, it is necessary to assume conditions in regard to the life of the plant and its operation, which, in practice, from the very nature of a public utility property, cannot be realized. These assumptions are as follows:

(1) That the physical plant of the utility, or at least its component parts, have a definite life;
(2) That the value of the plant must have been returned to the owners at the end of this period;
(3) That the owners must reinvest this returned capital at the end of the period in a new plant.

These assumptions are fundamentally wrong. It is impossible to give exactly the life of any parts of a utility; therefore, it is impossible to set aside or accumulate exact amounts to replace them. The best that can be done is to assume some term of years, based on former experience with the physical element under consideration. In addition, new developments, discoveries, and inventions, may, almost over night, cause valuable machinery and equipment to be consigned to the scrap heap, and thus the element of obsolescence comes into being. Conditions may change slowly or suddenly, so that the equipment of a utility becomes inadequate to handle the conditions as they

now exist. Thus, inadequacy is "developed." As an example of the former, the machinery of the cable roads in New York City had to give way to electric traction long before it was worn out. To illustrate the latter, there is the necessity for larger cars to handle the increasing traffic in large cities, or the necessity of railroads replacing wooden cars with steel, before the former have served their physical period of usefulness.

If the value of the plant was to be returned to the owners at the end of its period of usefulness, there would be a time, between the end of the old plant and the beginning of operation of the new, when there would be no utility. A utility, however, cannot go out of business of its own volition. It must continue to give good and useful service to the community (see Weatherly *v.* Capital City Water Co., Ala. 22 So., 140; Judge Haight in People *ex rel.* Manhattan Ry. Co. *v.* Woodbury, 203 N. Y., 231; 96 N. E., 420).

Therefore, in regard to depreciation, obsolescence, and inadequacy, such elements in the maintenance of any public utility should be taken care of by a fund maintained for this express purpose. This fund should be accumulated from a yearly increment taken from gross receipts and charged to maintenance costs. With a very few exceptions, every element in a utility begins to depreciate from the moment the construction forces place it in position. In a general sense, obsolescence and inadequacy are also depreciation. The utility should be kept up to date and up to 100% efficiency and value by constant withdrawals from this fund, to make good the three classes of deterioration previously mentioned. Therefore, at no time should such a fund amount to a very large percentage of the cost of the plant.

In order to maintain the fund in an equable manner, the amounts placed to its credit must, as far as possible, be evenly distributed over the years of the operation of the utility. There is no reason, however, why such amounts may not vary from year to year within reasonable limits, or why some definite plan should be fixed on and adhered to for a term of years, in order to accumulate such a fund, without any regard to conditions and circumstances which may develop or change in regard to the actual physical property.

It is impossible always to have an exact balance between the actual depreciation on the one hand, and the amount of the fund on the other. The fund, however, should be kept as near as may be to such

an amount. If such a fund is actually kept in trust for this purpose, there is no question but that it should be treated as an item of the plant, and rates should be allowed on it. However, if no such fund exists, or if it exists only in theory on the books of the corporation, then, in a valuation of the utility, depreciation may become a very important item, and must be given careful consideration. This may be treated in the valuation in one of two ways, which lead to the same result: The property may be appraised at its full cost, without depreciation, and returns may be allowed on this figure, no special mention being made of the depreciation fund; or the property may be appraised at its cost, less depreciation, and returns may be allowed on this and also on the depreciation fund.

That utility corporations are expected to keep the plant in first-class condition by constant withdrawals from some such sinking fund or its equivalent, and without allowing any excessive accumulation of such a fund, is set forth in Knoxville *v.* Knoxville Water Co., 212 U. S., 1; Cumberland Telephone and Telegraph Co. *v.* City of Louisville, 187 Fed., 637; and People *ex rel.* Manhattan Railway Company *v.* Woodbury, 203 N. Y., 231.

5.—Going Concerns and Franchise Values.—There are no other matters in the appraisal of a public utility that are so difficult of a fair and equable solution as the values to be given to the intangible elements, going concern and franchises. More difference of opinion exists among appraisers in relation to the method of treatment of these items than in regard to any others of the entire subject, some maintaining that no value whatever should be placed on them, others saying that as well not value the utility at all, as to omit them.

As to going concern value, there is nothing tangible that has ever entered into it, except the general management of the company. On the other hand, however, it is an admitted fact that the mere cost of the bare physical property of a utility does not represent the full value of a concern that is organized, managed, and operated in an efficient and satisfactory manner.

Various attempts have been made to arrive at some equable method of obtaining an equivalent tangible value for this intangible element, the best known of which is the capitalization of earnings over and above a fair return on the actual invested capital. In a physical valuation for the determination of rates, as the going concern value

deduced by this method depends on net income, and net income depends on rates, which is the very thing that is to be determined, it is impossible to obtain any such value in this way. Any attempt to deduce going concern value from rates, or to found rates on going concern value, is simply reasoning in a circle.

If this question is referred to the private business for study and comparison, the writer thinks that an equable solution of it may be deduced; and, as far as he knows, such solution has never been propounded before.

The private individual never charges up any value for going concern on the credit side of his balance sheet. What he does is this: In order to determine a price at which he can dispose of his commodity profitably, he first figures the total actual cost to produce the article (including amounts to cover depreciation, etc.), or, in other words, the "operating cost." The next item he adds to this cost is a fair rate of interest on the actual cost investment in the plant, and then follows an item to cover the general risk of the business. Lastly, he would add a figure to cover the fact that he is in business at all, or, in other words, that his concern is a "going" one, with all the advantages that this term implies. This, coupled with the risk item, is his profit in the venture.

There is no valid reason why the returns to the owners of a public utility corporation should not be determined in the same way. The operating expenses, including an allowance for depreciation of all kinds, are known or estimated, the physical cost has been determined, and a rate of interest must be allowed on this tangible value, such as the same amount of cash would return if placed at ordinary rates of interest. In addition, a further rate must be allowed to cover the risk of the enterprise.

Now, coming to the fact that the utility corporation is a going concern, in exactly the same way as the private enterprise is, there is no reason why the return for this intangible element cannot be equably obtained by adding some percentage to the figure already determined to cover the items before mentioned. In other words, these two items—the risk and the going concern—are the "profits" of the enterprise, and are so intimately related in practice as to be almost incapable of separation. In one sense, ordinary interest cannot be considered a profit, as such return could be obtained without any

investment in the utility. If there is no going concern, there is no profit, and, therefore, the return for going concern is covered when a fair rate (profit), over and above enough to cover expenses and ordinary interest, has been allowed.

Objections may be made to this method on the ground that it is impossible to determine any standard figure by which to gauge all cases in which going concern value must be determined, and that it really amounts simply to the opinion of the appraiser as to what to use. Such an objection, however, applies with equal force to any system whatever for estimating the value of going concern. In any event, the determination of a figure is simply a matter of the personal opinion of the appraisers as to what is an equable value for this element, and it is a much simpler matter to decide on a percentage of the actual physical investment in the plant, to cover it, than to go through an intricate set of calculations which are purely arbitrary and only tend to befog the real issue in the case.

If a valuation is made for the purpose of issuing securities, inasmuch as securities are issued, at least theoretically, to obtain capital to build and operate the utility, and going concern value cannot enter the problem until after the utility is in operation, therefore, no consideration should be given to this item for such purposes.

It is often assumed that franchise value and going concern value are so closely related as to make it impossible to separate one from the other for purposes of valuation. There is no doubt that these two elements are very intimately connected, but, at the same time, the writer thinks that there is enough material difference to make it very apparent why the two should be separated for purposes of discussion. A franchise is simply a permit, or the grant of the right, to conduct the business at all. On the other hand, going concern value is a value resulting from the use and application of the rights and privileges granted by the franchise. A concern may possess a franchise, which in itself is valuable, but on account of mismanagement and incompetency, the business may have no going value.

If a franchise only runs for a limited number of years, it has less and less value the nearer its time of expiration approaches. When it has expired, of course it has no value, although, under these conditions, the Courts have held that there is still a going concern value

in the property. (National Water-Works Co. *v.* Kansas City, 62 Fed., 853; Omaha *v.* Omaha Water Co., 218 U. S., 180.)

Furthermore, there may be a tangible element of value that attaches to a franchise from the fact that money may have been expended in a perfectly legitimate way to take the necessary steps to obtain it; or a franchise may have been purchased from a municipality, or by one company from another company. Thus, there may be a tangible element to take into consideration in the valuation of a franchise that does not occur in going concern value.

Although all franchises were granted by the State, the consensus of all Court rulings is that, once the grant having been made, and the corporation having expended capital under the implied protection of such grant, the State cannot revoke the franchise without giving to the corporation full and just compensation for its value. Just what value this is, no one has yet been able to determine. Every case has simply been decided by recourse to some arbitrary figure.

As far as rates are concerned, as the franchise is only the permit to conduct the business, if a fair return has been allowed for risk and going concern, that is all that is just and equable, and no special provision should be made for the franchise value, except in so far as actual expenses have been incurred in obtaining the same.

6.—Valuation for Purchase or Sale.—If a utility is to be valued for the purpose of purchase or sale, either between companies or between a company and a community, in the absence of a special agreement to the contrary, the present or fair value (in the majority of cases, cost of reproduction less depreciation) is the just and equable figure to be used for this purpose, except as hereinafter noted. Actual cost should be used simply as a guide in arriving at this figure.

By referring back to the paragraphs dealing with the sale of a private business, and applying the same reasoning to the utility corporation, the writer thinks that the justice of this method under these conditions will be perfectly apparent. The security holders have a perfect right to have the full value of the plant, as it stands at the time of sale, returned to them, as this is the figure it would cost the purchaser to reproduce it. Conversely, if the plant is now worth less than when it was built, the purchaser is entitled to the reduced price, as he could now build a similar plant for this figure. No purchaser has any grounds to say to a seller that he must part with his

goods at a less price than they would bring in the open market; nor has the seller a right to expect a higher price under similar conditions.

There are some exceptions to this use of the present value for some items, in cases where a community is the purchaser and the seller is a corporation that has exercised its functions under a charter granted by said community. These items are those of going concern and land values.

In regard to going concern value, the doubt as to the justice of allowing an amount to cover this item under the foregoing conditions arises from the fact that the community generally claims that, the franchise having expired, or being about to expire, and the corporation being about to go out of business, there is no value left for going concern. On the other hand, the corporation claims that, as the community is going to obtain a plant fully developed and in full operation, it is very different from starting up a new enterprise from the beginning, and that there is, therefore, a perfectly legitimate value to going concern.

The truth of the matter is that both parties to the controversy are partly right and partly wrong, and the only equable solution is to follow a middle course and allow some value for the going concern, but not such a large figure as the corporation would undoubtedly ask. Following such a course is the way the Courts have attempted to decide this difficult question (National Water Works Co. *v.* Kansas City, 62 Fed., 853).

As to land values, here also, under similar conditions of sale, it may not be equable to charge the full reproduction cost. The reasons for this in the case of land are perhaps not as apparent as in that of going concern, but the following are set forth as some limiting considerations:

If a utility is being appraised for purposes of sale, it may be that although the land now has a high value, other land could be obtained for the same purpose at a greatly reduced price, that would be just as satisfactory; but, as the utility cannot be moved without the destruction of the plant, it is impossible to separate the land from the utility. Furthermore, the land may have been obtained by recourse to the law of the right of eminent domain, and owners thus forced to part with their property against their will, when they may have realized that they were losing a good investment. It does not seem

equable that they or their immediate descendants should be compelled to pay a greatly increased price under such conditions.

In a case such as the foregoing, if the sale was one between private interests, it would simply mean that the buyer would go where cheaper land could be procured, if an exorbitant price were demanded by the seller. Both the buyer and seller are free agents, and are not compelled to enter the transaction against their will. In the case between the community and the public utility company, however, as the latter is and has been a monopoly, and for good and valid reasons either one or both of the parties may have been compelled to enter the transaction against their will, the solution of this part of the problem must be treated along somewhat different lines from that of other tangible property. There is only one seller and only one buyer, and no open market.

Under such circumstances, each case must be solved according to its own merits. It would seem here, as in the case of going concern values, that a middle course must be followed which will give as equable a solution as possible to both parties.

7.—*Valuation for Taxation.*—For the purposes of taxation, it does not make any difference what value is used, if the utility is under full State control, except that it might favor one section of the country as against another—this applying more to the railroads than to any other utility.

This can be explained as follows: The utility being under State regulation, all operating expenses are allowed over and above any return on the investment, and taxes are classed as an operating expense. If a local section of a road is assessed at a high valuation for purposes of taxation, and most of the revenue is obtained from through traffic, this traffic would then have to pay these taxes, to the benefit of the locality; but if the utility is a local one, there is no difference one way or another. The taxes come from the people in the form of operating expenses, and are returned to them as taxes, be the assessed value high or low, or the taxes on the assessed value high or low.

IV.—Conclusion.

There are some phases of the public utility corporation that the writer has not considered it necessary to discuss, inasmuch as they have no direct bearing on the valuation of the property of these com-

panies. Such, for example, is the company organized simply with the idea of acquiring and holding the securities of public utility corporations, now known as a "holding company," which class of concern is at present being severely criticized by the public and the States. A number of questions, also, that are being debated at the present time, such as the agency theory as applied to corporations in their relations to the State, and whether a public utility has the right to invest surplus profits, or a depreciation or amortization fund, in new construction and additions to the plant, and then charge rates on such investment, have not been touched upon. In addition, some physical elements, such as piecemeal construction, solidification and adaption of roadbed, pavement over mains, working capital, etc., have not been discussed in detail. The reasons for these omissions are that it is impossible to discuss every question that may have a bearing on the physical valuation of the property of public utility corporations, within the limits of this paper.

Furthermore, no attempt has been made to quote all legal decisions that may apply to the subjects discussed, nor has any attempt been made to give decisions in support of all contentions and arguments. Only enough have been given to show the general trend in the more important cases.

As was stated in regard to the private business, so also every public utility is capable of endless modifications and variations in its details of organization, the conduct of its affairs, and its relations to and with the State. When all is said and done in regard to the control and valuation of these corporations, every case must finally be decided on its own merits, and with due regard to the conditions and circumstances immediately surrounding it.

The real object of a valuation of any public utility is to obtain a figure such that any calculations or regulations founded on it as a basis will be just to all parties that have any interest whatsoever in the matter.

In the foregoing pages, the private business, the private corporation, and the public utility corporation have been outlined and compared; and, based on the similarity and the dissimilarity of such corporations with the private business, it has been shown how the property of these companies may be valued for different purposes. Broadly speaking, as far as the actual physical business is concerned, there is no difference

between the public utility corporation and the private business, and it is for this reason that a physical valuation of the property of a company of this class should follow the general lines that a private business would use in its accounting systems. There is no reason why there should be any difference in this respect between the two, except where elements are introduced into the problem that arise from the monopolistic character of the public utility.

On the other hand, however, it has been shown how the public utility corporation as an organization is, in its very inception, entirely different from the private business and the private company, and, on account of these methods of organization and development, how its relations with the State are of necessity on an entirely different status from those of the latter concerns. These main points of difference are here summarized as follows:

1.—The public utility corporation, before engaging in business at all, must obtain special favors from the State.

2.—It enters into direct business relations with the State.

3.—It therefore becomes the agent of the State.

4.—After having once entered into such an agreement with the State and accepted the trust, although it now has invested its own capital (property) in the enterprise, it cannot withdraw from the agreement on its own volition.

5.—Such invested property is held under different conditions and circumstances than that invested in private enterprises.

6.—It is a monopoly granted and, with certain limitations, upheld by the State.

7.—It is for these reasons that the State has the right to exercise detailed control over the public utility.

Therefore, in making an appraisal and valuation of the property of public utilities, if the points of similarity between this business and the private business and private corporation are kept constantly in mind, it will be seen why actual cost to date is the basic figure to be used in all cases, as has been set forth in the foregoing paragraphs; and, if this figure is used, it eliminates, as has been shown, many of the elements in such work that seem to be so difficult of solution. Actual cost is a figure equable to the investor (the real owner), to the promoters, to the managers; and last, but by no means least, such figure is absolutely just and equable to the public at large.

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PAPERS AND DISCUSSIONS

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INVESTIGATION OF THE PERFORMANCE OF A REACTION TURBINE.*

By R. L. Daugherty, Esq.†

Synopsis.

A careful and accurate test of a 550-h.p. reaction turbine recently installed at Cornell University showed a very high efficiency. In addition to the regular load test at constant speed, the torque, head, and discharge were measured at zero speed, and readings were also taken of speed, head, and discharge under no load. From these data it was possible to construct, at least approximately, a complete characteristic curve for the turbine.

1.—Cornell University generates its power for lighting and other purposes in a hydro-electric power plant which operates under a head of approximately 140 ft. The water is led from Beebe Lake to the power-house through a pipe line 5 ft. in diameter and 1 711 ft. long. When the present power-house was built, in 1905, it was equipped with four Pelton impulse wheels, but the increasing demands for power have made it necessary to add to the capacity of the plant. After due consideration of the various factors involved, it was decided to put in a reaction turbine for the additional unit.‡

*This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

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‡"Choice of Turbine for University Power House", by R. L. Daugherty, *Sibley Journal of Engineering*, Vol. 28, p. 191.

2.—The turbine selected was made by the I. P. Morris Company, of Philadelphia. It is rated at 550 h.p., at 600 rev. per min., under a head of 142 ft. The runner is 27 in. in diameter. The turbine is direct-connected to a 450-kv-a., 2 300-volt, 3-phase, 60-cycle alternator, supplied by the General Electric Company.

3.—The new turbine was tested by the writer in February, 1914. In this work he was ably assisted by Mr. F. G. Switzer, a graduate student and Fellow in Sibley College. The latter calibrated all the electrical instruments used and took charge of the determination of the generator output and efficiency.

4.—With the data secured in this test, it was possible to construct curves which would show the operation of the turbine under any combination of head, power, and speed. This is rarely done except in the case of a brake test. On account of the favorable conditions for testing, it is believed that a higher degree of accuracy than usual was attained. The efficiency of the turbine itself was found to be very high.

5.—The method of estimating the head in a turbine test is a matter of agreement in any case. The head consumed by the wheel alone is the fall from the surface of the head-water to the surface of the tail-water, minus all losses in the penstock and draft-tube. It is usually obtained by taking the difference in the heads at the entrance to and exit from the turbine. Let the pressure, when reduced to feet of water, be denoted by p, the elevation in feet above any arbitrary datum plane by z, and the velocity, in feet per second, as determined by the cross-sectional area and the measured rate of discharge, by v. If subscript $(_1)$ be used to indicate values at the point of entrance to the turbine case and subscript $(_2)$ corresponding values at the point of discharge from the turbine into the draft-tube, the head on the turbine will be:

$$h = z_1 + \frac{v_1^2}{2g} + p_1 - z_2 - \frac{v_2^2}{2g} - p_2.$$

6.—The draft-tube is often regarded as an integral part of the turbine, and thus any losses connected with it are to be charged to the turbine. Thus, the head on the turbine and draft-tube will be the fall from the head-water to the tail-water, minus the loss in the penstock

FIG. 1.—CORNELL UNIVERSITY POWER-HOUSE.

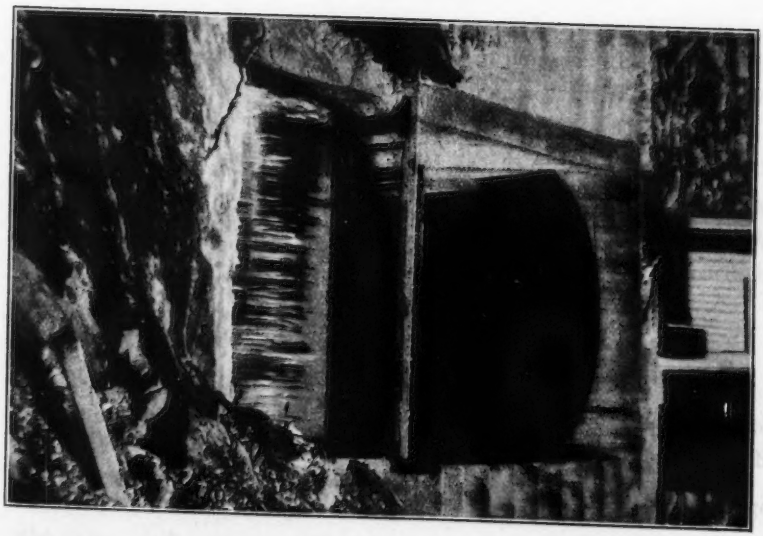


FIG. 2.—WEIR IN TAIL-RACE.

only; or, it is obtained in a test, if z_1 is the elevation of the point where the pressure is measured above the tail-race, by

$$h = z_1 + \frac{v_1^2}{2g} + p_1.$$

This value of h will be greater than that given by the equation in Paragraph 5. The difference between the two is equal to the friction head in the draft-tube plus the velocity head lost at the discharge from the mouth of the draft-tube. To illustrate this it might be noted that for Run 7 the head computed by the method shown in Paragraph 5 was 137.9 ft., and, as computed by the method just given, it was 140.46 ft. Throughout this paper the latter method will be used.

7.—In the test the pressure at the intake was measured with a mercury column built in the form of a U-tube. The height of the mercury columns was read on a metal scale graduated to hundredths of a foot. The oscillation of the columns was usually less than 0.010 ft., so that accurate readings were readily secured. There were four pressure connections equally spaced around the circumference of the penstock, and the true pressure was taken as the average of the four readings. It was noted, however, that there was very little difference in the pressures as measured at the four points.

8.—For convenience, the pressure was transferred to the center of the shaft which was 3.82 ft. above the zero point on the metal scale. If the two mercury column readings be denoted by R_1 and R_2, the pressure transferred to the center of the shaft will be

$$13.57\ (R_1 - R_2) - (3.82 - R_2).$$

The vertical distance from the center of the shaft to the crest of the weir was 14.0 ft. If the head of water over the weir be noted by H, the elevation of the center of the shaft above the tail-race level will be

$$14.0 - H.$$

The sum of these two expressions will be the value of

$$z_1 + p_1.$$

The intake pipe at the point where the pressure was measured is 30 in. in diameter. From this area and the measured rate of discharge, the value of v_1 may be obtained. By adding $\frac{v_1^2}{2g}$ to the above, we get the value of the head on the turbine.

9.—The rate of discharge was measured with a suppressed weir in the tail-race. A steel plate with a sharp beveled edge in good condition was set in just before the test. The length of the crest was 7.573 ft., and its height above the bottom of the channel was 2.83 ft. A set of baffles was inserted just below the draft-tube, and a raft was floated on the water to kill the surface ripples. The head on the weir was measured with a hook-gauge. The zero was set by a surveyor's level and checked by an observation when the water was just ready to flow over the weir. At the conclusion of the test, the zero was again checked by the level. During the test there was a small constant leakage through the other wheels. This was determined by a leakage run before the test was begun.

10.—The rate of discharge was computed by the modified Francis formula:

$$q = 3.33\, L \left(H + \frac{v^2}{2g}\right)^{\frac{3}{2}}$$

where,

q = cubic feet per second;
L = length of crest, in feet;
H = head on weir, in feet; and
v = velocity of approach, in feet per second.

If the velocity of approach is uniform over the entire cross-section of the channel, this formula is reasonably correct. In many cases, however, the actual velocity near the surface is much greater than the mean velocity, and there is practically a dead-water space near the bottom of the channel. Thus the effective velocity head is greater than that determined from the mean velocity. As v is obtained by dividing the rate of discharge by the cross-sectional area of the channel, it is customary to multiply $\frac{v^2}{2g}$ by a factor which may range from 1.0 to 2.0, according to circumstances. In this case a traverse of the channel with a current meter, by E. W. Schoder, Assoc. M. Am. Soc. C. E., and Professor Turner, showed a uniform distribution of velocity, and thus justified the use of a factor of unity.

11.—This uniform distribution of velocity is probably due to the fact that the distance from the draft-tube to the crest of the weir is only about 30 ft. The draft-tube is vertical, and discharges its water

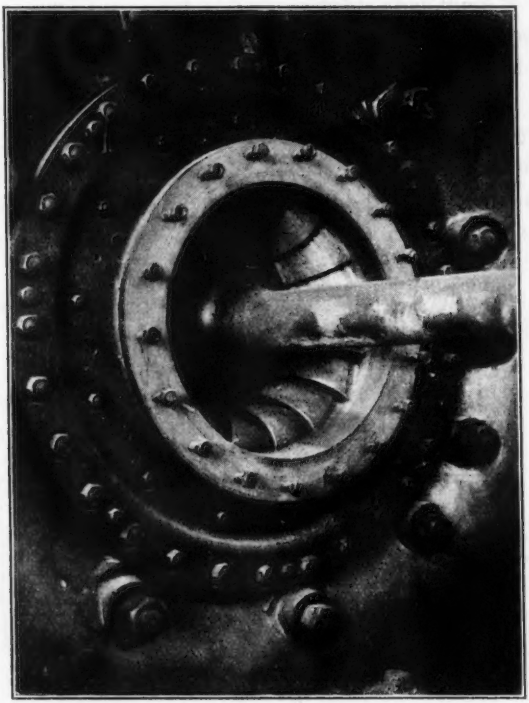


FIG. 3.—REACTION TURBINE, WITH DRAFT-ELBOW REMOVED, SHOWING RUNNER.

FIG. 4.—WATER RHEOSTAT.

directly against the bottom, thus it is probable that there would be a high bottom velocity for some distance. If the tail-race had been much longer we might then have expected to find the more usual (that is, unequal) distribution of velocities.

12.—The power output of the generator was absorbed by a water rheostat suspended in a pool in the creek. This rheostat consisted of three pipes at the vertices of an equilateral triangle, with sheet-iron plates bolted to them. The load was varied by changing the depth of immersion. A similar rheostat, used in some testing in 1913, proved to be very satisfactory, giving a perfectly steady load. The one used in this test was designed to carry twice as much load as the former instrument, but it failed to do so without modification. The design of such a rheostat seems to be largely a matter of trial, in view of the limited data available and also on account of the important factor of

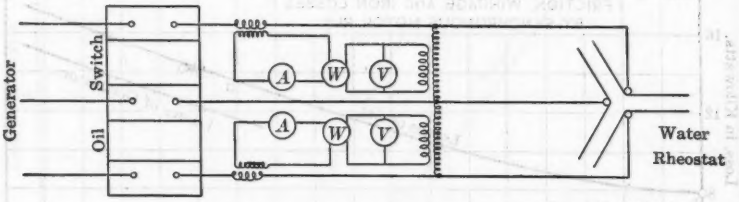


DIAGRAM OF CONNECTIONS

FIG. 5.

the purity of the water.* At the time of the tests in 1914, the run-off in the creek was mostly from melting ice and snow and therefore purer than the ground-water encountered in 1913.

13.—In order to make the rheostat carry the load, it was necessary to bend the plates so as to bring them within 2 in. of each other. It is probable that the fluctuations on the heavier loads were due to the boiling of the water between the plates. By taking readings every 2 min. during quite a period of time, it is believed that a good average was obtained, despite this slight unsteadiness. The accuracy of the results is roughly checked by the fact that the power factor remained approximately constant for all the loads.

14.—All electrical instruments used were carefully calibrated by comparison with standards in the possession of the Physics Department of Cornell University. The ratios of current and voltage trans-

*See *Power*, August 4th, 1914, p. 184.

formers were determined as well as the phase angles of the latter. The phase angles of the current transformers were found to be negligible.

15.—At the end of each run the resistance of the armature was determined by the drop-of-potential method. The friction, windage, and iron losses were found by operating the generator as a synchronous motor. It was also desired to find the value of the friction and windage alone, and to determine how these losses varied with the speed. As this could not be done with the synchronous motor run, a small, direct-current motor was belted to the set. The input to the motor was read, and the motor losses were deducted. If the power of this motor had been sufficient to drive the set at 600 rev. per min.,

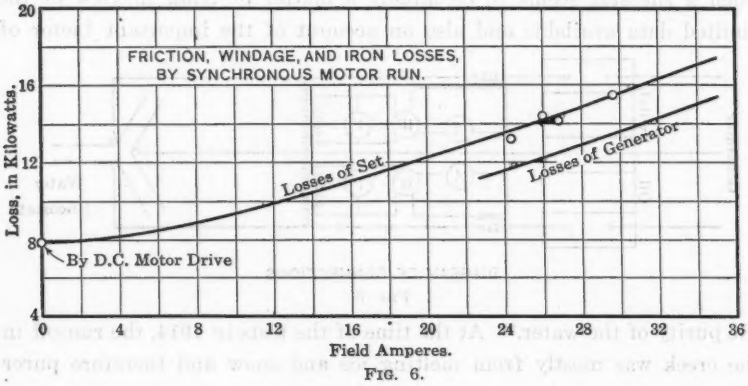


FIG. 6.

under full field, the synchronous motor run would have been unnecessary. The two combined, however, gave everything that was desired. (Figs. 6 and 7.) The friction and windage of the generator was arbitrarily assumed to be three-fourths of that of the set.

16.—In the test, runs were made under various loads at a constant speed of 600 rev. per min. The turbine was also prevented from rotating, and the torque at various gate openings was measured by a system of levers and standard weights. The gate-openings were the same as those used in the load runs. Readings were also taken of head and discharge. In addition, the turbine was permitted to run at run-away speed, the only load on it being the friction and windage of the generator. For the same gate openings, readings were taken of speed, head, and discharge.

17.—The usual commercial test, made at a constant speed and under an approximately constant head, is quite valuable, yet it is not entirely satisfactory. The question is always open as to what the effect would have been if the speed had been a little higher or a little lower. Also, in many plants, where the speed of the turbine must necessarily be constant, the head will vary through quite a range, due

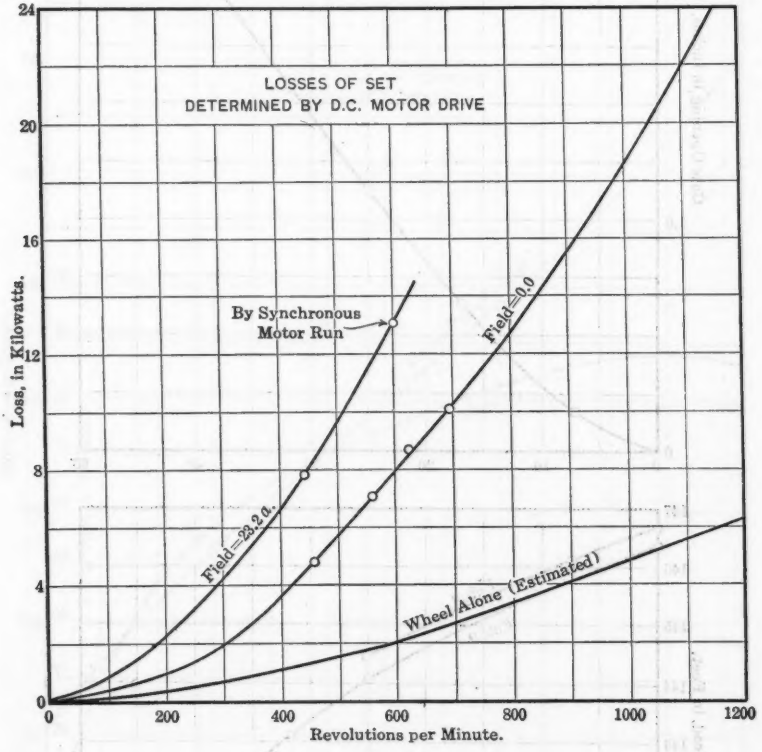


FIG. 7.

to fluctuations in the stream flow. Thus, in actual operation the relation between speed and head may not always be the best. In order to determine fully the characteristics of a given turbine, it is necessary to test it at various gate openings under several relations of speed to head. It is clear that the speed may be varied, as in the usual brake test, or it may be possible to vary the head. Either method will accomplish the desired result. In the test under discussion the head

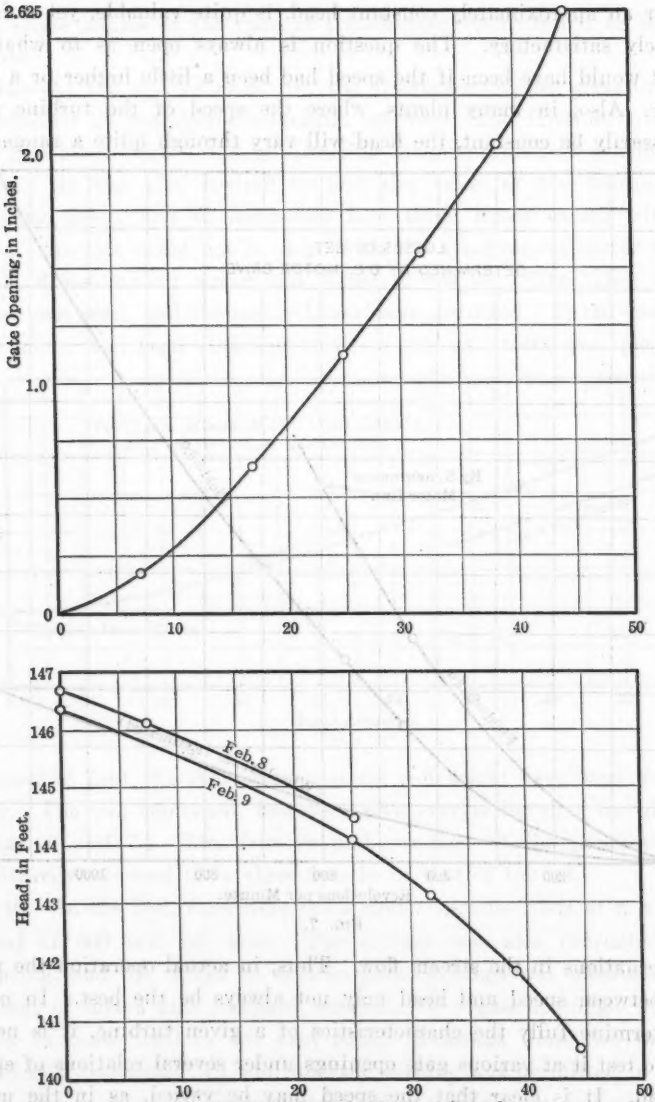

2.625
2.0
1.0
0
Gate Opening, in Inches.
0
10
20
30
40
50
147
146
145
144
143
142
141
140
Head, in Feet.
Feb. 8
Feb. 9
0
10
20
30
40
50
Discharge, in Cubic Feet per Second.

FIG. 8.

was throttled down by the gate-valve to about two-thirds of its normal value. A load run was then made under the regular speed of 600 rev. per min. It was hoped to secure a number of such points at several

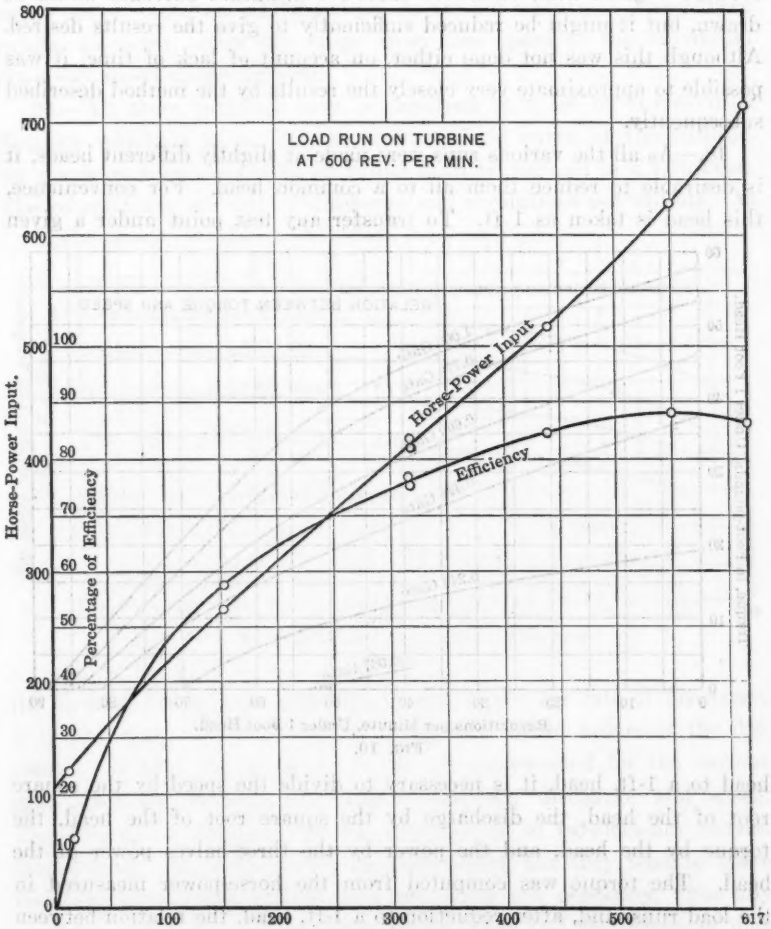


FIG. 9.

gate-openings and at several values of the head, but lack of time available for the test prevented. By throttling the head, while keeping the speed constant, we obtain values higher than normal of the ratio

of speed to head. In order to obtain values of this ratio lower than normal, the head could be made the maximum value possible and the speed of the generator reduced. Of course, the speed could not be reduced a great deal, otherwise excessive armature currents would be drawn, but it might be reduced sufficiently to give the results desired. Although this was not done either, on account of lack of time, it was possible to approximate very closely the results by the method described subsequently.

18.—As all the various runs were made at slightly different heads, it is desirable to reduce them all to a common head. For convenience, this head is taken as 1 ft. To transfer any test point under a given

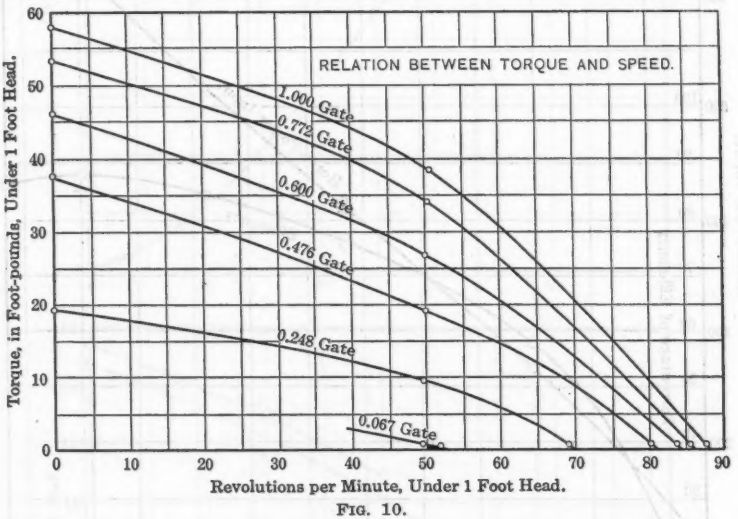


FIG. 10.

head to a 1-ft. head, it is necessary to divide the speed by the square root of the head, the discharge by the square root of the head, the torque by the head, and the power by the three-halves power of the head. The torque was computed from the horse-power measured in the load runs, and, after reduction to a 1-ft. head, the relation between torque and speed is obtained, as shown in Fig. 10. The relation between rate of discharge and speed is shown in Fig. 11.

19.—The relation between torque and speed, or discharge and speed, is not a straight line, but, if a suitable scale is chosen, may be represented by a fairly flat curve. With the three points determined for

each gate opening, it is possible to draw curves which cannot be very far from correct, assuming that the three points are accurately determined. If additional points had been determined by the methods previously outlined, there would then have been no question about the entire curve; but these additional points are not essential. These three points, moreover, cover the range from zero speed to the maximum run-away speed under no load.

20.—All these relations may be involved in a set of curves in a diagram which is known as the characteristic curve of the turbine. To construct such a diagram all values must be reduced to a 1-ft. head. The co-ordinates are rate of discharge and revolutions per minute. As

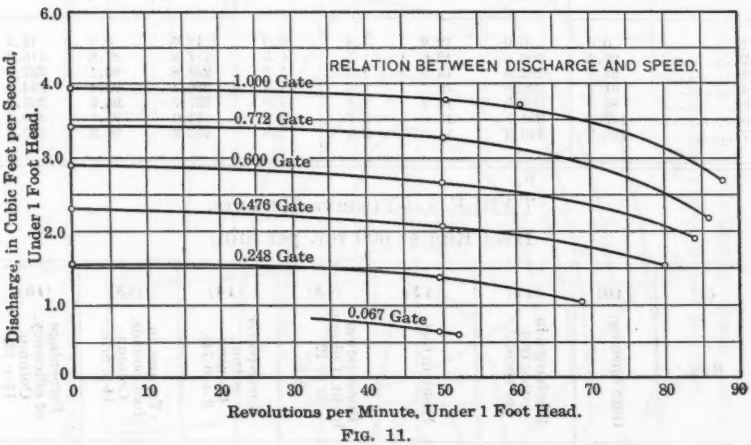


FIG. 11.

the horse-power input is directly proportional to the rate of discharge, it is customary to add a scale for it, parallel to the values of the discharge. Within this diagram, curves are constructed for the various gate openings, showing the relation between discharge and speed. The diagram is completed by the construction of iso-efficiency curves. Occasionally, iso-power curves are also added, but they tend to make the diagram too complicated, and are really not necessary. For any conditions of operation, the horse-power input and the efficiency may be readily noted on the diagram. The product of the two gives the horse-power developed. This system of plotting a turbine performance was devised by D. W. Mead, M. Am. Soc. C. E., and is given in his "Water Power Engineering".

21.—Accepting the curves in Figs. 10 and 11 as correct, the efficiency may be computed at any intermediate point, and thus the characteristic curve may be constructed, as shown in Fig. 12. Such a diagram covers every possible condition under which the turbine may operate.

TABLE 1.—GENERATOR RESULTS.

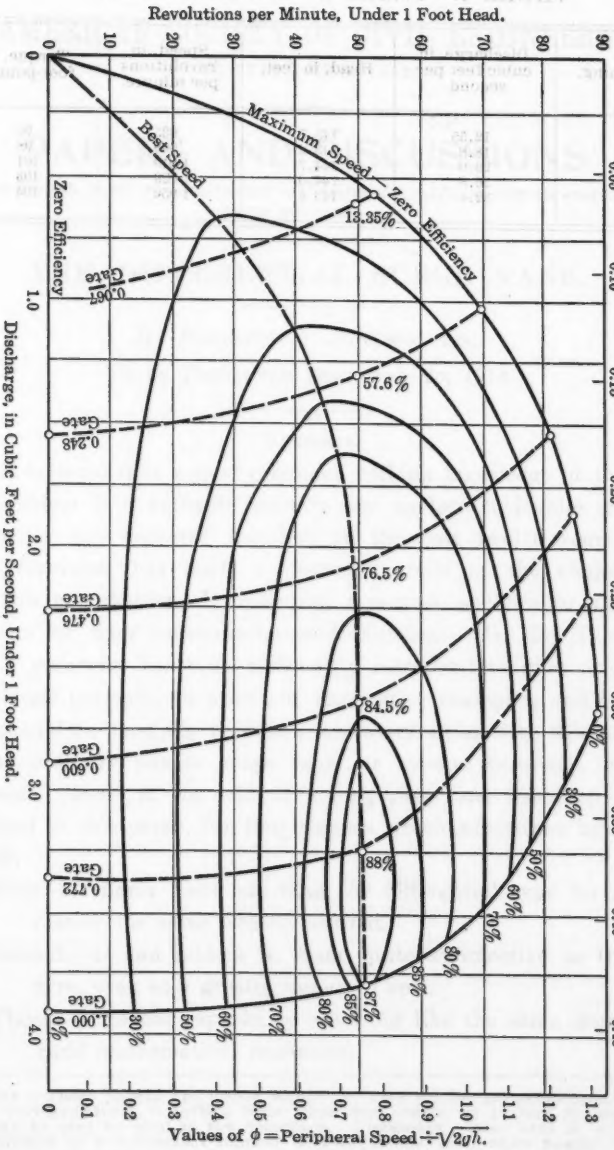
(1) Run.	(2) Average amperes.	(3) Kilowatt output.	(4) Friction, windage, and iron loss.	(5) Input to field.	(6) Armature resistance loss.	(7) Total kilowatt input. Columns 3 + 4 + 5 + 6.	(8) Percentage of generator efficiency. Column 3 ÷ 7.	(9) Kilowatt input from turbine. Columns 3 + 4 + 6.
1	0.0	0.0	12.2	1.4	0.0	13.6	0.0	12.2
2	29.2	103.4	12.6	1.5	0.3	117.8	87.8	116.3
3	59.6	222.6	13.2	1.7	1.3	238.8	93.1	237.1
4	61.7	220.8	12.7	1.6	1.3	236.4	93.4	234.8
5	86.5	309.9	13.2	1.8	2.8	327.7	94.6	325.9
6	107.6	391.8	13.8	2.0	4.4	412.0	95.1	410.0
7	125.5	440.3	14.0	2.1	5.9	462.3	95.2	460.2

TABLE 2.—TURBINE RESULTS.
Load Run at 600 rev. per min.

(1) Run.	(10) Gate opening.	(11) Discharge, in cubic feet per second.	(12) Head, in feet.	(13) Horse-power input. Column $\frac{11 \times 12}{8.8}$	(14) Horse-power output. Column 9 ÷ 0.746.	(15) Torque, in foot-pounds. Column 14 × 8.75.	(16) Percentage of efficiency. Column 14 ÷ 13.
1	0.067	7.4	146.1	122.8	16.4	143.5	13.35
2	0.248	16.4	145.4	271.0	156.0	1 365.0	57.6
3	0.476	25.4	144.3	417.0	318.0	2 780.0	76.3
4	0.476	25.0	144.2	410.0	315.0	2 750.0	76.7
5	0.600	31.8	143.1	517.0	437.0	3 820.0	84.5
6	0.772	38.8	141.8	625.0	550.0	4 820.0	88.0
7	1.000	44.5	140.5	710.0	617.0	5 400.0	87.0

TABLE 3.—TURBINE AT ZERO SPEED.

Gate opening.	Discharge, in cubic feet per second.	Head, in feet.	Torque, in foot-pounds.
0.248	18.7	144.9	2 760
0.476	27.5	143.9	5 390
0.600	34.8	142.8	6 550
0.772	40.2	141.8	7 520
1.000	46.3	140.6	8 130

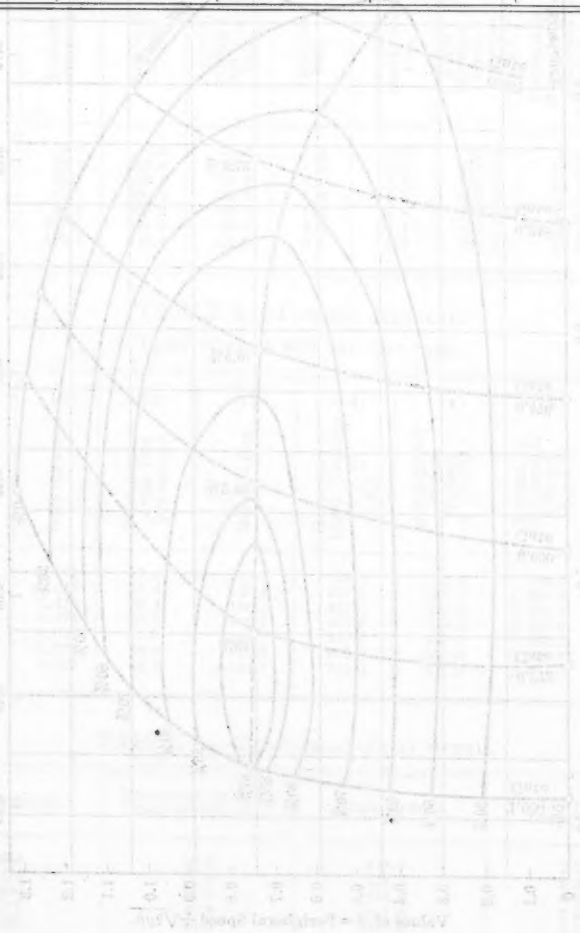


CHARACTERISTIC CURVE OF 27-INCH I. P. MORRIS TURBINE, TESTED UNDER 142 FEET HEAD.

FIG. 12.

TABLE 4.—Turbine at Run-Away Speed.

Gate opening.	Discharge, in cubic feet per second.	Head, in feet.	Speed, in revolutions per minute.	Torque, in foot-pounds.
0.248	12.55	145.7	835	90
0.476	18.8	145.2	969	98
0.600	23.0	144.7	1 010	101
0.772	26.8	144.0	1 029	103
1.000	32.4	143.4	1 052	104



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THE DIFFERENTIAL SURGE TANK.

By Raymond D. Johnson, Esq.

To be Presented December 2d, 1914.

Synopsis.

It is believed that a good practical working knowledge of the surge tank problem is a valuable asset to the modern hydraulic engineer. The writer has gathered together, in the past twelve years, many useful formulas, has made a thorough study of the subject, and takes this opportunity of presenting a concise analysis to his fellow engineers for their consideration and criticism. Mr. Roy Taylor, the writer's assistant, has been sufficiently interested to give up months of his time patiently to work out illustrative examples, and his criticism should be carefully read as a necessary adjunct to this paper.*

The ordinary simple surge tank, or storage reservoir, which is commonly placed at the end of a long pipe line, has been entirely neglected in this paper, for four reasons which experience has demonstrated:

First.—It never costs less than the differential type for approximately the same benefit realized.

Second.—It can seldom be made quite so effective as the latter type, even at a greatly increased cost.

Third.—It is not capable, to anything like the same degree, of a rigid mathematical treatment.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* For Mr. Taylor's discussion, see p. 2555.

Fourth.—It is subject to unstable conditions of the vibrating water column, to an extent which renders an exact prediction of its behavior, if not impossible, at least far more difficult than that of the differential type.

The differential surge tank consists of a stand-pipe of about the same diameter as the conduit, freely connected to it, and a storage tank of larger dimensions, surrounding the stand-pipe and connected to the conduit by a properly restricted passage.

In a simple tank, the level of the stored water, following a demand for more power, represents the accelerating level which is urging more water from the forebay, and measures the head acting on the water-wheel. In the differential type, owing to the resistance interposed between tank and conduit, the level of the stored water is quite independent of the acceleration, and does not affect the water-wheel governor directly. The water in the stand-pipe takes care of these things, and acts like a simple tank of small dimension which is supplemented by the steadying action of the stored water, fed into the system in an independent, non-synchronous manner, meeting all demands for water without causing the unstable, pendulum-like behavior which is so characteristic of the simple surge tank.

On account of the possibility of a rigid mathematical treatment of the ideal or theoretical differential action, several interesting facts are discoverable which would probably not have been generally demonstrable by a study of the so-called simple tank, which, by the way, is anything but simple in all respects save as to the conception of its physical structure. Some of the features most noteworthy, are:

a.—The working volume of a differential surge tank, for a given velocity change and for a given degree of stability in regulation, is independent of the length of the conduit, the area decreasing as the length of conduit, and consequent height of tank, increase.

b.—The highest possible surge above forebay level following a complete shut-down, is not necessarily due to the shutting off of the maximum velocity obtainable, but may occur as a result of throwing off part load at a critical velocity.

c.—It is possible, and often quite feasible, to design for a perfect dead-beat or non-oscillatory condition, where no unnecessary

motion of the water occurs, nothing more than a change of level corresponding to the inevitable gradient change.

d.—Every design is intimately associated with an infinite number of similar designs, so related, kinematically, that the one type may be varied indefinitely in its own class, without further computation, except of a primitive nature. This fact favors graphical methods, and enables all computations to be reduced to the utmost simplicity.

These features are here brought out, along with a general mathematical treatment deduced from fundamental principles of mechanics, and the illustrations by Mr. Taylor complete the demonstration of the differential principle, and show its advantages over the ordinary method now in somewhat general use.

Introduction.

The function of a stand-pipe or simple surge tank, to regulate a long enclosed column of water feeding water-wheels, is well enough understood, and many authors have written on this subject since the presentation of the writer's paper entitled "The Surge Tank in Water Power Plants."* Therefore he will pass over the elementary matter, involving description of the undulating water wave produced by load changes, and take up the more technical side of the question, illustrating some exact mathematical methods of arriving at intelligent results in design. In the following studies the exact determinations are confined to the first quarter cycle of the water wave, that is to say, the period during which the first drop or rise of water within the tank takes place, up to the time when a reversal of motion of the water begins.

The differential surge tank or regulator, as has been described in the writer's previous paper, consists of a simple stand-pipe freely connected to the water-power conduit, and supplemented by a tank surrounding the stand-pipe and communicating with the conduit through a definitely restricted opening. This restricted passage offers a resistance to the flow of water back and forth through it, and, when properly designed and proportioned, has a remarkably beneficial effect on surges which would otherwise attain harmful, if not dangerous, proportions in a simple surge tank of much larger dimensions.

* *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, p. 443 (1908).

No one has yet succeeded in writing correct equations for the phenomena during the first quarter cycle in a simple surge tank, when friction, velocity head, and governor action are considered. Neither is it possible to do this for the differential regulator, except for the case when the stand-pipe area approaches zero for a surrounding tank of a given size. A certain capacity of stand-pipe is essential for many reasons, as will appear later; one of its principal benefits is to furnish a little time for the pressure change following a load change, so as not to interfere with the governor and throw additional duty on the fly-wheel. It is highly instructive and very satisfactory to be able to write perfectly exact equations for all the phenomena which take place during the first quarter cycle, taking into account the friction in the conduit and eliminating without error the action of the governor. This can easily be done when the capacity of the internal stand-pipe approaches zero, and the subsequent addition of this capacity and the further refinement of the area of the restricted passage for an actual design become an easy task for a little judgment assisted by arithmetic integration.

An undulation or water wave once set in motion by a load change may have any one of four characteristics, depending on the size of the tank, other conditions being fixed.

(1) It may die out as a damped harmonic;

(2) It may live indefinitely at the same amplitude;

(3) It may increase in amplitude with each oscillation;

(4) It may continue to drop, theoretically, without turning around to rise again, if the acceleration of the conduit is not fast enough to catch up to the increasing draft velocity as the head recedes and the governor opens up the water-wheel wider and wider.

(This last case is interesting only from a mathematical standpoint, and will not here be further considered.)

(5) As a modification of Case (1), it may die out with no oscillation at the end of the first quarter cycle in infinite time. This case is called the non-oscillatory or dead-beat condition, where perfect or absolute damping takes place, and the water does not pass beyond the normal quiescent level due to the velocity required by the suddenly applied load.

When load changes happen to occur in step with the natural period of vibration of the water column, Case (1) usually takes on the characteristics of Case (3), and experience has shown that this test, applied to many simple tank designs, disqualifies them for safe use. On the other hand, it is extremely rare to hit upon a case where a differential regulator will not hold the water within definite limits up and down, however long synchronous load changes be applied. Under these conditions the vibration may sometimes increase for one or two swings, after which it will remain at constant amplitude for an indefinite number of continued synchronous load changes. The writer recently ran across a practical case where a simple tank of certain dimensions took on the characteristics of Case (3), following a single load change, and the same tank, converted into a suitable differential regulator, successfully withstood the test, even, of synchronous load changes. Case (5) is in most cases economically out of the question to obtain with a simple tank, but frequently for long pipe lines, large friction, or high velocities, or all of these, it becomes quite possible to design for the dead-beat condition with the differential principle.

One of the interesting paradoxes of this question is the fact that stable regulation, or freedom from vibration, becomes easier to obtain as the inertia effects get worse. In other words, a surge tank of given area will approach the dead-beat wave as the length of the pipe line and the velocity increase. The height of such a tank would have to increase with the length of the pipe line and with the velocity, but unless the area were correspondingly decreased, the oscillation would become steadier as the inertia effects became worse. The reason for this is that the work which may be accomplished within a surge tank is proportional to the square of the height through which the water may be permitted to vary, and this height, for equivalent stability, may vary directly as the friction gradient, which itself varies directly as the length of the pipe line and about as the square of the velocity. The work required to be done, however, varies only directly as the friction gradient, which is to say, also, as the length of the pipe line and as the square of the velocity. Hence, doubling either of these quantities would double the permissible height for water variation within the tank, and would thereby multiply its capacity for work by four. This brings out forcibly the important fact that

very long pipe lines need not be avoided through any misconceived fear of dangerous oscillations of the water column. The unavoidably large change in gradient following a load change is, to be sure, a somewhat undesirable attribute of a very long pipe line, because it causes the governor to travel a little farther than it would if the head did not vary, but no more serious effect than this comparatively mild one need be apprehended if the regulator is designed intelligently.

Some people have argued against the differential action because it causes a more rapid initial variation in head than the simple surge tank, but the fact is that the small capacity furnished by the internal stand-pipe of the differential regulator is quite sufficient to afford just as good practical results as a simple tank of like dimensions, and the manifest advantages in all other respects ought to make it the preferred type in all cases. The writer has never yet found a condition where he could see any advantage in the use of a simple surge tank, and long ago he abandoned the idea entirely, as always extravagant, often impracticable, and sometimes dangerous.

Another important function of the internal stand-pipe is to afford a relief overflow into the tank in case of a sudden shut down, which causes the water to rise immediately to the top of the stand-pipe, and, while spilling into the tank, the conduit receives the benefit of this suddenly applied back pressure which cuts down to a minimum the total quantity of water to be caught. For ordinary load changes, the water merely rises or falls a moderate distance within the stand-pipe, without, of course, overflowing, and the effect of the restricted opening is to force a more rapid variation of level in the stand-pipe than takes place in the surrounding tank, but the restriction is ordinarily not sufficient to cause a greater total variation of level in the stand-pipe than subsequently takes place in the tank due to the transfer of water through the restricted passage. The object of this restriction is to cause a greater acceleration than would result from the comparatively sluggish movement of the water surface in a simple tank, thereby minimizing the quantity of water to be handled by the tank and greatly increasing its ability to check or choke harmful and unnecessary surges. The size of a simple tank to produce the same net results as to regulation of the water-wheel is sometimes enormously larger than that of a differential regulator, the exact

ratio depending on the conditions to be met, and this is especially evident when the test of synchronous load changes is applied.

The ideal differential action to which the following equations apply needs a careful description in order to forestall any misconception as to the limitations of the mathematics, and to set forth clearly the general fundamental conditions which make possible at all a rigid mathematical treatment of the surge phenomena. Certain assumptions are necessary, which are not strictly adhered to in the final design but may be modified with little additional labor when the action of the regulator is fully understood.

First.—The stand-pipe area is considered to be so small that a variation in water level within it takes place in a negligible time compared to the remaining time in which complete acceleration of the water column is effected.

Second.—The restricted opening is assumed to vary in size according to certain laws, with the result that, when a sudden change in level takes place within the stand-pipe (due to a sudden load change), the water level remains absolutely stationary at the new position until complete acceleration is effected in the conduit.

Third.—The size of the tank is required to be such that the ultimate change in water level in one direction shall be exactly equal to the sudden change in the stand-pipe for the given load change under consideration.

Fourth.—The inertia of the water in the regulator itself is neglected.

Fifth.—Velocity changes only, rather than load changes, are considered, and therefore the head, H, is not involved in the theory; it enters only in an extraneous computation to convert velocity change into load change.

Sixth.—The friction in the stand-pipe, velocity head, and the weir head when spilling over the top, are neglected.

Seventh.—The kinetic energy in the flowing water is computed from the average velocity, $\frac{Q}{A}$, and is not refined to take into account the slight error introduced by reason of the non-uniform velocity existing in the cross-section of a cylinder of flowing water.

The first assumption is never actually realized, except where compressed-air tanks are used.

The second assumption is seldom, if ever, actually necessary, because, as will appear later, a constant size of choke hole is quite adequate for practical results, especially when assisted by some capacity in the stand-pipe which has to be neglected for the sake of the integral calculus.

As has been stated before, the following mathematical treatment in its entirety deals only with the phenomena which take place during the first quarter cycle of the water wave; that is to say, during the period of the first programme of acceleration, positive or negative, while the water level in the tank is changing in one direction and that in which it has been started by a load change. The subsequent plotting of the various waves of pressure and velocity must be carried on by recourse to the principles of arithmetic integration which any one who has the patience and is sufficiently interested may do with no more than an ordinary understanding of the fundamental laws of Newton and a good training in the rapid use of a slide-rule. It is not the present purpose to describe these methods in detail, because satisfactory results may be obtained in many ways, and the individual may often select the method best suited to his mind. Suffice it to say that although the mathematical treatment herein set forth is as complete as the writer has been able to make it, yet when the design of a regulator is approximated as closely as possible from the available equations, it is usually much more satisfactory and profitable to one lacking experience, to integrate the phenomena second by second in the common-sense way than to rely solely on judgment for the effect of the departures which have to be made from the hard and fast mathematical assumptions and results.

The ideal differential action to which the equations apply is necessarily confined to one particular load change, and if mathematically ideal conditions were required for a greater load change, the size of the choke hole would have to be increased. It follows, therefore, that the design should be made ample in the first place for the greatest load change for which good regulation is desired, and this should be controlled by the size of the greatest ordinary daily sudden variation when the operation of the plant is regular and normal. If there occurs a greater load change than that for which the regulator has been designed, the result will be a greater total variation of pressure level within the stand-pipe than is necessary or than subsequently takes

place in the tank, and this condition is not desirable as an ordinary operating feature. If, however, abnormal conditions obtain for a time, such as synchronous load changes or excessively large ones, this disproportionate variation in water level within the stand-pipe is an excellent thing, because it serves to counteract and deaden any tendency toward an increasing or unstable vibration of the water level. In other words, the differential action should be harmless under normal conditions, if properly designed, and should be particularly beneficial for abnormal conditions. The advantage of this principle, as distinguished from the simple tank, is not nearly so apparent during the first quarter cycle as it is later in the history of the wave, but even at first it has one decided advantage in that the total variation of head on the wheel is considerably less than in a simple tank of equal size; this permits a smaller water-wheel to adhere to the load following a load change without reaching full gate, or conversely, permits a wheel of the same size to deliver the new increased load at smaller gateage, which means at better efficiency if near full gate, and which, therefore, means less acceleration required in the long conduit, and better conditions all around.

Proper characteristics of the water-wheel itself are of great importance in assisting the surge tank, and particular attention should be paid to the water-wheel specifications, if the surge tank is not to be unreasonably taxed; and, if the wheels are already installed when the task of providing a suitable regulator is undertaken, a knowledge of the water-wheel characteristics is essential for intelligent design. While the efficiency is increasing with the load, regulation is comparatively easy, but, beyond the crest of the efficiency curve, the regulator has additional work to do, and in some instances of excessive friction in a long conduit, further opening of the gates may actually produce less power, which is a case impossible for any regulator to improve.

Nomenclature.—The nomenclature herein adopted is as follows:

t = time in general;
t_a = time of acceleration;
t_r = time of retardation;
T = time to complete one quarter cycle of the water wave;
L = length of the conduit;
A = area of the conduit;

F = net area of tank, that is, in excess of the stand-pipe area;

a = area of restricted opening or port with 100% coefficient of discharge;

a_0 = area of restricted opening when $t = 0$;

a_1 = area of restricted opening when $t = T$;

v = velocity in the conduit;

v_1 = initial conduit velocity before acceleration begins ($t_a = 0$), or = conduit velocity when $t_r = T$, or = constant-draft velocity between surge tank and water-wheel, in terms of the conduit velocity during retardation;

v_2 = conduit velocity when $t_a = T$, or = initial conduit velocity before retardation begins ($t_r = 0$), or = constant-draft velocity between surge tank and water-wheel in terms of the conduit velocity during acceleration;

c = a coefficient such that cv^2 = total losses of head in the conduit (these losses may or may not include the velocity head, depending on the location of the surge tank);

y = departure of the water level in the tank from its initial quiescent position previous to a load change;

$y_1 = y$ for $t = T$ = also, by hypothesis, the amount of the initial sudden change of level in the stand-pipe;

p = percentage of velocity change = $\frac{v_2 - v_1}{v_2}$ in acceleration;

$r = \frac{v_1}{v_2} = 1 - p$;

k = stability factor of the water wave = $\frac{y_1}{c\,(v_2^2 - v_1^2)}$;

log. = natural logarithm, where not specified;

$$Z = \sqrt{\frac{y_1}{c} + v_1^2}, \text{ or } y_{1_a} = c\,(Z^2 - v_1^2);$$

$$Z_1 = \sqrt{v_2^2 - \frac{y_1}{c}}, \text{ or } y_{1_r} = c\,(v_2^2 - Z_1^2);$$

$$Z_0 = \sqrt{\frac{y_1}{c} - v_2^2}, \text{ or } y_{1_r} = c\,(v_2^2 + Z_0^2);$$

$$X = \frac{Z}{v_2} = \sqrt{k\,(1 - r^2) + r^2}.$$

ACCELERATION.

By hypothesis we have, for positive acceleration,

$$dt = \frac{\frac{L}{g}\, d\, v}{y_1 - c\,(v^2 - v_1^2)}$$

from which, by integration between the limits v_1 and v, we have the general expression for the time which has elapsed in acquiring any velocity

$$t_a = \frac{L}{2\,g\,c\,Z} \log. \frac{(Z - v_1)\,(Z + v)}{(Z + v_1)\,(Z - v)} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Multiplying both sides of the differential equation by $A\ v$, we have

$$A \int_0^t v\, dt = \frac{A\ L}{g} \int_{v_1}^{v} \frac{v\, dv}{y_1 - c\,(v^2 - v_1^2)},$$

which equals the total quantity of water discharged through the conduit in the time, t. Now, by hypothesis, the total quantity drawn through the water-wheels in the time, t, is $A\ v_2\ t$, and the difference must be taken from the tank; whence,

$$A\ v_2\ t_a - \frac{A\ L}{g} \int_{v_1}^{v} \frac{v\, dv}{y_1 - c\,(v^2 - v_1^2)} = y\, F.$$

Integrating and simplifying, and substituting the value of t_a from Equation (1), we have

$$y = \frac{A\ L}{2\,g\,c\,F} \left\{ \frac{v_2}{Z} \log. \frac{(Z - v_1)\,(Z + v)}{(Z + v_1)\,(Z - v)} - \log. \frac{Z^2 - v_1^2}{Z^2 - v^2} \right\} \ldots (2)$$

This is the general expression giving the momentary relation between the drop in water level in the tank and the corresponding conduit velocity which has been attained.

When v becomes equal to v_2, y becomes equal to y_1, if F is of the correct size, hence we may place $y = y_1$, and solve for F. Remembering that

$$T_a = \frac{L}{2\,g\,c\,Z} \log. \frac{(Z - v_1)\,(Z + v_2)}{(Z + v_1)\,(Z - v_2)} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

we have, for correct size of tank,

$$F = \frac{A\ L}{2\,g\,c\,y_1} \left\{ \frac{v_2}{Z} \log. \frac{(Z - v_1)\,(Z + v_2)}{(Z + v_1)\,(Z - v_2)} - \log. \frac{Z^2 - v_1^2}{Z^2 - v_2^2} \right\} \ldots (4)$$

or, in other notation,

$$F = \frac{A\ L}{2\,g\,c^2\,v_2^2\,k\,(1 - r^2)} \left\{ \frac{1}{X} \log. \frac{(X - r)\,(X + 1)}{(X + r)\,(X - 1)} - \log. \frac{k}{k - 1} \right\} (5)$$

from which F may be solved directly for any velocity change and any degree of stability, k, desired. Equation (5), at first sight, may seem to be a useless algebraic contortion of Equation (4), but it may be found instructive to have due regard to k rather than the more direct value y_1 which in itself gives no adequate idea of how much the motion of the water level exceeds the gradient change.

The head, h_p, acting on the restricted passage or "port" at any time $= y_1 - y$, and may be expressed in terms of the corresponding velocity as follows:

$$y_1 - \frac{A\,L}{2\,g\,c\,F}\left\{\frac{v_2}{Z}\log.\frac{(Z-v_1)(Z+v)}{(Z+v_1)(Z-v)} - \log.\frac{Z^2-v_1^2}{Z^2-v^2}\right\}$$

or, substituting the value of y_1 in terms of Z from Equation (4), we have

$$h_p = \frac{A\,L}{2\,g\,c\,F}\left\{\frac{v_2}{Z}\log.\frac{(Z-v)(Z+v_2)}{(Z+v)(Z-v_2)} - \log.\frac{Z^2-v^2}{Z^2-v_2^2}\right\} \quad \ldots(6)$$

The quantity of water necessary to be forced through the port by the head, h_p, is $A\,(v_2 - v)$; therefore,

$$a\,\sqrt{2\,g\,h_p} = A\,(v_2 - v)$$

and

$$a = \frac{\sqrt{A}\,(v_2-v)}{\sqrt{\frac{L}{c\,F}\left\{\frac{v_2}{Z}\log.\frac{(Z-v)(Z+v_2)}{(Z+v)(Z-v_2)} - \log.\frac{Z^2-v^2}{Z^2-v_2^2}\right\}}} \quad \ldots(7)$$

which is the general expression for the size of the port for any assigned conduit velocity, during a surge.

When $v = v_1$ at the beginning of the cycle,

$$a_0 = \frac{A\,(v_2-v_1)}{\sqrt{2\,g\,y_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(8)$$

When $v = v_2$, the expression becomes indeterminate, but when evaluated by differentiating the numerator and denominator, we find

$$a_1 = \left\{\frac{A\,F\,y_1}{L}\left(1-\frac{1}{k}\right)\right\}^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(9)$$

which is always less than a_0.

It is interesting to note the extreme simplicity of the foregoing equations when there is considered to be no friction in the conduit. A single port of fixed area is then found to be exactly adequate to hold the accelerating water level at one point while the conduit velocity

is attaining a value which satisfies the new load. In other words, when the tank has the correct area, F, the pressure head on the fixed port which exists at any instant is just that required to force through it the difference between the quantity of water demanded by the water-wheel and that being furnished, at that particular instant, by the conduit.

By hypothesis, we have

$$dt = \frac{L}{g\,y_1}\,dv \text{ and } dy = \frac{A}{F}\,(v_2 - v)\,dt.$$

Eliminating dt, we have

$$dy = \frac{A\,L}{F\,g\,y_1}\,(v_2 - v)\,dv,$$

from which, by integration between the limits v_1 and v, we obtain

$$y = \frac{A\,L}{2\,g\,F\,y_1}\left\{(v_2 - v_1)^2 - (v_2 - v)^2\right\} \ldots\ldots\ldots\ldots (10)$$

also, similarly,

$$t = \frac{L}{g\,y_1}\,(v - v_1) \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

$$y_1 = \frac{A\,L}{2\,g\,F\,y_1}\,(v_2 - v_1)^2 \text{ and } T = \frac{L}{g\,y_1}\,(v_2 - v_1) = \sqrt{\frac{2\,L\,F}{g\,A}}$$

$$F = \frac{A\,L}{2\,g\,y_1^2}\,(v_2 - v_1)^2.$$

These equations hold good for both acceleration and retardation. Expressing the area of port in terms of v and y we have, as before,

$$a^2 = \frac{A^2\,(v_2 - v)^2}{2\,g\,(y_1 - y)},$$

or, substituting the values of y_1, and y in terms of v,

$$a^2 = \frac{A\,F\,y_1\,(v_2 - v)^2}{L\left\{(v_2 - v_1)^2 - (v_2 - v_1)^2 + (v_2 - v)^2\right\}} = \frac{A\,F\,y_1}{L} = \text{constant},$$

or,

$$a = \sqrt{\frac{A\,F\,y_1}{L}} = \text{constant}. \ldots\ldots\ldots\ldots\ldots (12)$$

Equation (12) is the limit of Equation (7) as c approaches zero.

The value of a_0 for $c = 0$ may also be shown to be the same, by substituting for $(v_2 - v_1)$ in Equation (8) its value as just determined in terms of y_1, and we again have

$$a_0 = \sqrt{\frac{A\,F\,y_1}{L}}.$$

Also, since $\frac{1}{k}$ becomes zero, Equation (9) becomes

$$a_1 = \sqrt{\frac{A\,F\,y_1}{L}}.$$

The port area remains constant throughout the quarter cycle.

Equations (10), (11), and (12) are useful when c is so small as to make the previous formulas too sensitive to solve with ordinary logarithmic tables. Equation (12), as compared with Equation (7), shows that the less the friction the less the variation in a in order to hold a perfectly stationary accelerating water level in the stand-pipe. This variation in the size of effective port may be approximated as closely as desired by a series of holes up and down the stand-pipe, which would shut off automatically as the water dropped below them in the tank, but this refinement is ordinarily not warranted, and when there is some appreciable capacity in the stand-pipe, as for example, if it has an area $= A$, a single port of fixed size is quite sufficient for all practical purposes except for very unusual conditions. No serious error would be made in fixing the size of the port by Equation (8), but a trial computation by arithmetic integration for any particular case would show that a somewhat smaller hole than a_0 gives slightly better results. The reason it is not usually necessary to refine too carefully the dimensions of the port, is that, if one uses Equation (8), the port will always be somewhat too large, which is the same thing as saying that a somewhat larger velocity change than the one assumed would most nearly approach the mathematical ideal. No particular harm results from making the hole a little too large, although it should be as nearly right as possible, for as it gets larger the dangers of the simple tank are being approached.

When k approaches unity, Case (5), or the dead-beat wave, is approximated, and for $k = 1$, the water level does not pass beyond the final quiescent position for the increased load.

When $k = 1$, $Z = v_2$, and Equation (1) becomes for dead-beat conditions,

$$t_a = \frac{L}{2\,g\,c\,v_2} \log. \frac{(v_2 - v_1)\,(v_2 + v)}{(v_2 + v_1)\,(v_2 - v)} \qquad \ldots\ldots\ldots\ldots (13)$$

and when $v = v_2$, $T_a = \infty$.

Equation (2) is much simplified by substitution of $Z = v_2$ and combining the logarithms, and becomes, for dead-beat conditions,

$$y = \frac{A\,L}{g\,c\,F} \log. \frac{v_2 + v}{v_2 + v_1} \qquad \ldots\ldots\ldots\ldots\ldots (14)$$

and when $v = v_2$ and y becomes y_1, we have

$$y_1 = \frac{A\,L}{g\,c\,F} \log. \frac{2\,v_2}{v_2 + v_1} = c\,(v_2^2 - v_1^2)$$

from which

$$F_a = \frac{A\,L}{g\,c^2\,(v_2^2 - v_1^2)} \log. \frac{2\,v_2}{v_2 + v_1} \quad \ldots\ldots\ldots\ldots (15)$$

Also, when $k = 1$, $Z = v_2$ and Equation (7) becomes, for dead-beat conditions,

$$a = \frac{\sqrt{A}\,(v_2 - v)}{\sqrt{\frac{2\,L}{F\,c} \log. \frac{2\,v_2}{v_2 + v}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Equation (8) remains unchanged, except for the possible substitution of $y_1 = c\,(v_2^2 - v_1^2)$; and Equation (9) becomes $a_1 = 0$.

It has been pointed out previously that the extent of the variation in a depends on the amount of the friction, and the greatest variation from a_0 to a_1 is found when $a_1 = 0$, for the condition of perfect damping effect. If this ideal result were required to be practically obtained with a tank as small as given by Equation (15), the standpipe would need to contain an infinite number of holes, which would merge into a slot of peculiar shape, but this is an unnecessary amount of refinement because absolute damping is very little better than a near approach to it, and, besides, a moderately larger tank could be made to accomplish it with far less refinement of the port area. One fixed hole should usually be sufficient in all cases.

The limiting value of F for small load changes is interesting, particularly because as v_2 approaches v_1 in value, the solution of previous equations becomes troublesome, and, for the complete plotting of a curve, for example, the limit must be known. It might appear at first thought that when the velocity change is zero the tank area would also be zero, but for a given required stability, k, the value of F approaches a perfectly definite limit as r approaches unity. The value of a, however, does approach zero. The limit of $\frac{(X - r)\,(X + 1)}{(X + r)\,(X - 1)}$, as r approaches unity, is $\frac{k}{k - 1}$; therefore Equation (5) is a true indeterminate with respect to F, and by multiplying numerator and denominator by X and performing the necessary successive differentiations we find

$$F_0 = \frac{A\,L}{4\,g\,c^2\,v^2\,k} \left\{ 1 - (k - 1) \log. \frac{k}{k - 1} \right\} \quad \ldots\ldots\ldots (17)$$

The value of F_0 for dead-beat conditions is also useful to assist in solving equations which approach the indeterminate form as k approaches unity.

The limit of $(k-1)\log.\frac{k}{k-1}$ as k approaches unity is zero, and therefore

$$F_{d_0}=\frac{A\,L}{4\,g\,c^2\,v^2}\qquad\ldots\ldots(18)$$

This completes the study for positive acceleration, so far as the writer has carried it up to the present time, and the variations which occur for retardation will now be considered.

Retardation.

By hypothesis, we have for retardation

$$dt=\frac{\frac{L}{g}\,dv}{y_1-c\,(v_2^2-v^2)}.$$

In the corresponding equation for acceleration the constant term in the denominator was always positive and hence the integration took but one form. In this case, however, y_1 does not always reach forebay level for part load rejected, and therefore may have a value either greater or less than cv_2^2, thus causing the constant term in the denominator to vary through zero from negative to positive as y_1 is increased.

(*1*) *When Water Does Not Reach Forebay Level on Rejection of Load.*—When $y_1 < c\,v_2^2$, we have, as for Equation (1),

$$t_r=\frac{L}{2\,g\,c\,Z_1}\log.\frac{(v_2-Z_1)\,(v+Z_1)}{(v_2+Z_1)\,(v-Z_1)}\qquad\ldots\ldots(19)$$

and, as a companion of Equation (2), we have, similarly,

$$y_r=\frac{A\,L}{2\,g\,c\,F}\left\{\log.\frac{v_2^2-Z_1^2}{v^2-Z_1^2}-\frac{v_1}{Z_1}\log.\frac{(v_2-Z_1)\,(v+Z_1)}{(v_2+Z_1)\,(v-Z_1)}\right\}\ldots(20)$$

and for the value of F we have

$$F=\frac{A\,L}{2\,g\,c\,y_{1r}}\left\{\log.\frac{v_2^2-Z_1^2}{v_1^2-Z_1^2}-\frac{v_1}{Z_1}\log.\frac{(v_2-Z_1)\,(v_1+Z_1)}{(v_2+Z_1)\,(v_1-Z_1)}\right\}.(21)$$

For the case when $y_1=cv_2^2$, which means that a certain critical value of F just forces the water to forebay level for an assigned value

of $v_2 - v_1$, we find that Equation (19) becomes indeterminate as Z_1 approaches zero. On evaluation by differentiation we have

$$t_r \text{ (for } Z_1 = 0) = \frac{L}{g\ c} \times \frac{v_2 - v}{v\ v_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

and Equation (20) becomes

$$y \text{ (for } Z_1 = 0) = \frac{A\ L}{g\ c\ F} \left\{ \log. \frac{v_2}{v} - v_1 \frac{v_2 - v}{v\ v_2} \right\} \quad \ldots\ldots\ldots (23)$$

$$\text{and } F \text{ (for } Z_1 = 0) = \frac{A\ L}{g\ c^2\ v_2^2} \left\{ \log. \frac{v_2}{v_1} - \frac{v_2 - v_1}{v_2} \right\} \quad \ldots\ldots\ldots (24)$$

When c approaches zero, the equations for retardation are exactly the same as already derived for acceleration, namely, Equations (10), (11), and (12).

The foregoing equations for retardation are chiefly interesting from a mathematical standpoint because in a practical case very little attention need be paid to what happens when part load is rejected, so long as a complete shut-down is properly taken care of. Therefore, the size of the port is controlled by the acceleration studies, and is limited in size only by the shut-down conditions, as will appear later. The equations for dead-beat conditions in retardation may as well be written, for the sake of comparison with Equations (14) and (15). By processes strictly analogous to the previous cases, we have for dead-beat retardation, which can exist only for rational values of $Z_1 = v_1$

$$y = \frac{A\ L}{g\ c\ F} \log. \frac{v_2 + v_1}{v_1 + v} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

and when $v = v_1$ and y becomes y_1, we have as compared to Equation (15)

$$F = \frac{A\ L}{g\ c^2\ (v_2^2 - v_1^2)} \log. \frac{v_2 + v_1}{2\ v_1} \quad \ldots\ldots\ldots\ldots\ldots (26)$$

It will now be seen that F in Equation (26) is always larger than F in Equation (15), because $\frac{v_2 + v_1}{2\ v_1} > \frac{2\ v_2}{v_2 + v_1}$, and, although full load may be thrown on without producing oscillation, in which case $\frac{2\ v_2}{v_2 + v_1}$ becomes $= 2$ for $v_1 = 0$, yet oscillation is bound to occur for full load rejected, except for an infinite value of F, because $\frac{v_2 + v_1}{2\ v_1}$ becomes ∞ for $v_1 = 0$.

In other words, the water must in all cases reach a level higher than the forebay. The value of F for dead-beat conditions for very small load changes, however, would be identical with that given by Equation (18), because the limit of log. $\frac{v_2 + v_1}{2 v_1} \div (v_2^2 - v_1^2)$ as v_2 becomes $= v_1$, is $\frac{1}{4 v_1^2}$.

When Water Rises Above Forebay on Rejection of Load.—The phenomena resulting from irrational values of Z_1 will now be taken up. This is by far the more usual case in practice, where the water rises above forebay level following a rejection of part load. As Z_1 becomes a surd, it is no longer of any use, and it must be supplanted by $Z_0 = \sqrt{\frac{y_1}{c} - v_2^2}$. Reverting to the original differential equation,

$$dt = \frac{\frac{L}{g} d v}{y_1 - c (v_2^2 - v^2)}$$

we find that when $y_1 > c v_2^2$, the integration takes the form of the tan.$^{-1}$, instead of the log., and we have

$$t_r = \frac{L}{g c Z_0} \left\{ \text{tan.}^{-1} \frac{v_2}{Z_0} - \text{tan.}^{-1} \frac{v}{Z_0} \right\} \ldots\ldots\ldots\ldots (27)$$

and

$$y = \frac{A L}{2 g c F} \left\{ \text{log.} \frac{v_2^2 + Z_0^2}{v^2 + Z_0^2} - \frac{2 v_1}{Z_0} \left(\text{tan.}^{-1} \frac{v_2}{Z_0} - \text{tan.}^{-1} \frac{v}{Z_0} \right) \right\} (28)$$

and

$$F = \frac{A L}{2 g c y_1} \left\{ \text{log.} \frac{v_2^2 + Z_0^2}{v_1^2 + Z_0^2} - \frac{2 v_1}{Z_0} \left(\text{tan.}^{-1} \frac{v_2}{Z_0} - \text{tan.}^{-1} \frac{v_1}{Z_0} \right) \right\} (29)$$

and evaluating Equation (29) for $Z_0 = 0$ we again have Equation (24), which shows that Equations (29) and (21) draw together, as they should, and become identical for the condition that forebay level shall exist in the tank when $t_r = T_r$.

The most useful equations for rejected load are those pertaining to a shut-down when $v_1 = 0$, and Equation (28) becomes

$$y = \frac{A L}{2 g c F} \text{log.} \frac{v_2^2 + Z_0^2}{v^2 + Z_0^2} \ldots\ldots\ldots\ldots\ldots (30)$$

and Equation (29) becomes

$$F = \frac{A L}{2 g c y_1} \text{log.} \frac{v_2^2 + Z_0^2}{Z_0^2} \ldots\ldots\ldots\ldots\ldots (31)$$

and, as $k_r = \frac{y_1}{c\, v_2^2}$, we may rewrite Equation (31) in terms of k_r as follows:

$$F = \frac{A\, L}{2\, g\, c^2\, v_2^2\, k_r} \log. \frac{k_r}{k_r - 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

from which k_r may be determined by trial, provided F has been previously fixed by the consideration of demanded load. It sometimes happens, however, that F is fixed by the maximum hydrostatic pressure desirable which determines k_r, in which case the value of a for demanded load is subject to adjustment to a given value of F.

The limiting size of the port hole, a_1, for a shut-down which obtains when $t_r = T_r$, just as the water comes to rest in its upward movement in the tank at the level of the top of the stand-pipe, is determined by processes analogous to those used in arriving at Equation (9) and becomes

$$a_1 = \left\{ \frac{A\, F\, y_1}{L} \left(1 - \frac{1}{k_r}\right) \right\}^{\frac{1}{2}}$$

or,

$$a_1 = v_2 \left\{ \frac{A\, F\, c}{L} (k_r - 1) \right\}^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

If k_r has been determined by trial from Equation (32), its value may be substituted in Equation (33) and the limiting value of a derived. The size of the port hole, as determined for demanded load, must not exceed this limiting value, for otherwise the water, having reached the top of the stand-pipe, following a shut-down, will not remain there for the time, T, but will shrink back down the stand-pipe during the quarter cycle, afterward rising again, and thus will produce a condition contrary to the hypothesis of a constant value of y_1, and may introduce an error of some magnitude. It happens, fortunately, that the requirement of design for excessively large demanded loads is rare, and almost always, when the value of a is decided on for demanded loads, it is incidentally smaller than the limiting value given by Equation (33). This will always be the case when, in the acceleration studies, $2\, r$ is not less than

$$c\, v_2 \sqrt{\frac{2\, g\, F}{A\, L}} + \frac{1}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

which has been determined by graphical methods, but should be verified by trial for any particular case. It will be observed that the less the friction, other things remaining unchanged, the greater the load change

for which the capacity of the port will provide without exceeding a suitable size for a shut-down; and when $c = 0$, the same value of a would be suitable, either for throwing on or off the full load. The factor, $\frac{1}{10}$, in Equation (34) is merely empirical.

In selecting the value of v_2, to determine the maximum height of the tank, it must be borne in mind that a larger value is likely to be required than the value of v_2 found for a maximum load demanded, because the conduit velocity somewhat exceeds the value existing when $t_a = T$, except for dead-beat conditions, and a shut-down could occur just at the critical point of maximum conduit velocity during the second quarter cycle following an increment of load. The extent of this excess may be left to the judgment, or it may easily be roughly determined by a few steps of arithmetic integration. It is really so small a matter as compared to the uncertainty attending the selection of a low enough value of c, that the judgment is usually quite sufficient, and, furthermore, it does not always follow that higher velocities call for higher surge tanks to catch all the water, for, as will now be demonstrated, there exists for all conditions, except the hypothetical one where $c = 0$, a critical velocity which necessitates a higher surge tank than any greater or smaller velocity. This critical velocity, of course, may be greater than that due to full load, but it frequently occurs at part load for large values of c or L, and in general for values of k_a near unity.

The height above forebay level to which the stand-pipe and tank must be carried to provide sufficient capacity to catch all the energized water following a shut-down, let us call d, and its value is evidently

$$d = c\ Z_0^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

and it is required to find the value of v_2 which makes d a maximum; and as c is constant for any particular case, d will be a maximum when Z_0 is a maximum.

Putting $y_1 = c\ (Z_0^2 + v_2^2)$ in Equation (31), we have

$$\frac{2\,g\,c^2\,F}{A\,L} = \frac{\text{log. } (v_2^2 + Z_0^2) - 2 \text{ log. } Z_0}{Z_0^2 + v_2^2}$$

from which we may find the value of v_2 when $\frac{d\,Z_0}{d\,v_2} = 0$, which we shall call v_c, and we have

$$v_c = \frac{1}{c}\left\{\frac{A\,L\,(e-1)}{2\,g\,F\,e}\right\}^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots\ldots (36)$$

in which e is the base of the natural logarithms and v_c becomes a little less than

$$\frac{1}{10\,c}\sqrt{\frac{A\,L}{F}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

If the value of v_2 which is chosen as a maximum to determine the greatest height of tank and stand-pipe is greater than v_c, we must conclude that a shut-down from some part load will require a higher elevation to avoid spilling. In many cases v_c may not be so much above v_2 as materially to increase the height of tank if the former value is assumed, even though we do not expect quite so high a value, and, where economically feasible, it is best to provide for the worst case chiefly because the absolute maximum velocity which may at some future time exist coincident with a shut-down is a difficult thing to fix with precision. It is usually economically feasible to do this when the value of c is high, as is commonly the case for very long pipe lines. When not commercially desirable to do this, the value of $\frac{v_2}{c}$ must be made large enough to cover all reasonable contingencies. The value of d for $v_2 = v_c$ may be expressed by solving for $d = c\,Z_0^2$ in Equation (31) and we have

$$d_{max.} = \frac{A\,L}{2\,g\,c\,e\,F} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

which is the greatest distance above forebay level which a tank of area F need ever be carried for any velocity whatsoever. Perhaps the most striking feature of this equation is its independence of the value of L. This is evident because c is proportional to L. The limiting value of a_1 for v_c is obtained from Equations (33), (36), and (38), namely,

$$a_1 \text{ (for } v_c) = \frac{A}{\sqrt{2\,g\,c\,e}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Equation (33) may also be written

$$a_1 = \sqrt{\frac{A\,F\,d}{L}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

This concludes the study in its present state of development, and it is to be hoped the critics will add something to it. Before closing it is desired to call attention to the kinematic relation which ties together all the foregoing equations.

For a given value of p and k in Equation (5), the number represented by $\frac{2\,g\,c^2\,v_2^2\,F}{A\,L}$ is constant $= N^2$ (say).....(41)

That is, if a certain fixed percentage velocity change, p, is desired together with an assigned fixed degree of stability, k, then any mutual interchange in the values of Equation (41) may be made without affecting the result. This is very important because each assignment of p and k covers an infinite variety of combinations without further computation except to manipulate Equation (41) in any conceivable manner so long as the ratio there expressed remains constant. As c varies approximately as $\frac{L^*}{A^{\frac{3}{5}}}$, we may more properly write, in this connection, $c = \frac{f\,L}{A^{\frac{3}{5}}}$ where f is the coefficient of friction, and Equation (41) becomes, omitting the constants, 2 g and f,

$$\frac{v_2^2\,L\,F}{A^{2.2}} = \text{constant.}$$

Also, similarly, $\frac{a\,\sqrt{L}}{A^{1.3}} = \text{constant}$, as $\frac{a\,\sqrt{c}}{A}$ is constant in Equations (8), (9), etc., for fixed values of p and k.

L and F may be varied inversely with respect to each other, v and A remaining constant, or any other similar interchange may be made without affecting the kinematic relation of the design to the assigned requirements.

As $L\,v^2$ is a measure of energy, it is apparent that the area of the surge tank varies inversely as the energy in the water column, other conditions remaining unchanged. If a design were contemplated where the values were as follows:

$$v_2 = 5,\ L = 5\,000,\ F = 1\,600,\ A = 100, \text{ and } a = 10,$$

a model on a small scale might be made which would give a good idea of the adequacy of the design, if the following values were used, keeping the total ratio $= 7\,962$ and 1.776 respectively,

$$v_2 = 2,\ L = 500,\ F = 10,\ A = 1.52, \text{ and } a = 0.137.$$

*This expression for the variation of c is not intended to represent an accepted law; it is merely illustrative. According to Williams and Hazen, for example, c would vary as $\frac{L}{A^{0.584} \times v^{0.15}}$, and some such suitable refinement would be desirable for actual use.

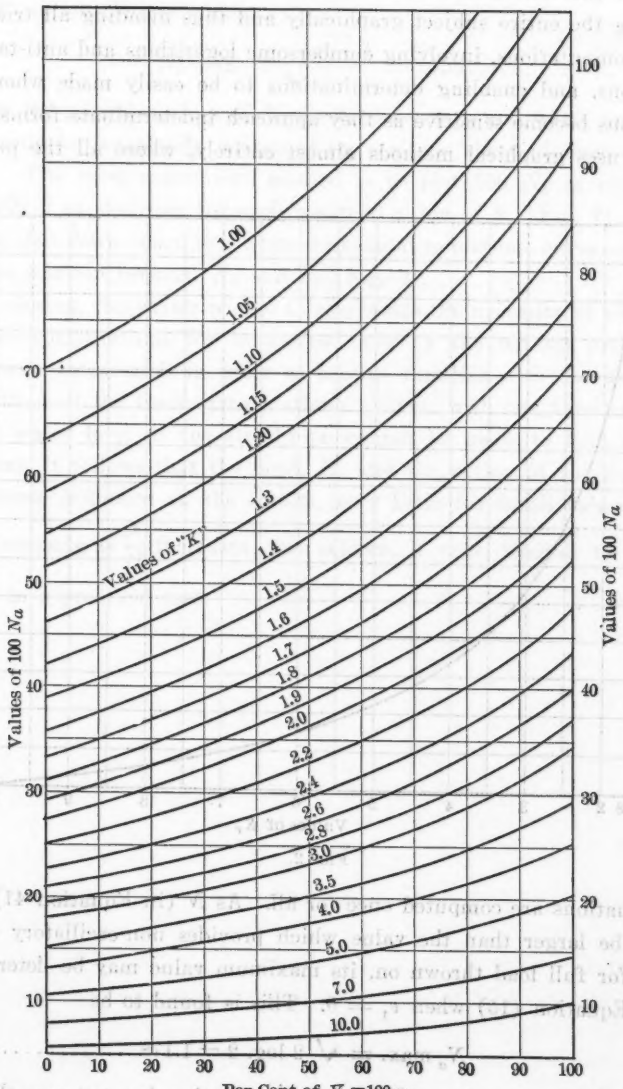
Values of "K"
1.00
1.05
1.10
1.15
1.20
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
2.2
2.4
2.6
2.8
3.0
3.5
4.0
5.0
7.0
10.0
Values of 100 N_a
Values of 100 N_a
Per Cent of $V_2 = 100\,p$
0 10 20 30 40 50 60 70 80 90 100
10 20 30 40 50 60 70 80 90 100

FIG. 1.

This interesting relation of scale suggests a convenient method of treating the entire subject graphically and thus avoiding all trial and error computations, involving cumbersome logarithms and anti-tangent functions, and enabling determinations to be easily made where the equations become sensitive as they approach indeterminate forms. The writer uses graphical methods almost entirely, where all the possible

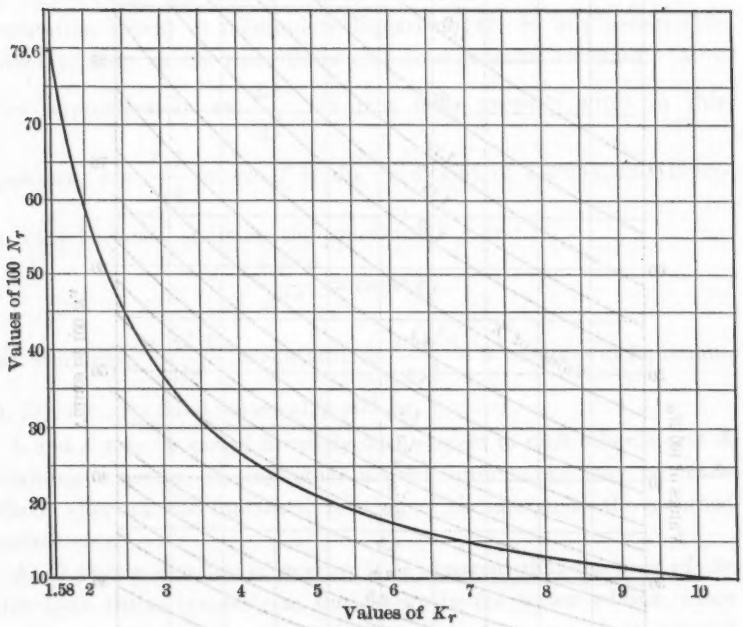

FIG. 2.

combinations are computed once for all. As N (in Equation 41) need never be larger than the value which provides non-oscillatory conditions for full load thrown on, its maximum value may be determined from Equation (15) when $v_1 = 0$. This is found to be

$$N_a \text{ max.} = \sqrt{2 \log. 2} = 1.178 \ldots\ldots\ldots\ldots\ldots (42)$$

Thus it is seen that the value of N in acceleration does not vary through a wide range, and all possible values may easily be contained on one sheet of cross-section paper, for they all lie between 0 and 1.178.

Similarly, for a shut-down, the value of N_r max. may be obtained from Equation (36) in which

$$N_r \text{ max.} = \sqrt{\frac{e-1}{e}} = 0.796 \ldots\ldots\ldots\ldots\ldots (43)$$

and all the possible values of N_r lie between zero and 0.796, and may also easily be completely contained on a single sheet of cross-section paper. The most convenient method is to plot 100 N_a as ordinates with 100 p as abscissas for each assigned value of k_a (Fig. 1). Also, for the shut-down chart, it is quite convenient to plot one curve, expressing the relation between N_r and k_r (Fig. 2).

In closing, the writer wishes to emphasize the necessity of performing the computations for numerous cases by the tedious process of arithmetic integration in order to become familiar with the uses and limitations of the foregoing equations. Since, with one fixed value of a, the water level in the stand-pipe cannot be made to hold exactly constant, it follows that the head, H, and the action of the governor have some influence on the results, very little for small values of N and increasing as $\frac{N}{H}$ increases, but seldom, if ever, leading to serious errors in a practical case.

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PENSTOCK AND SURGE-TANK PROBLEMS.

By Minton M. Warren, Esq.

To be Presented December 2d, 1914.

Synopsis.

Object.—The object of this paper is to give, in their best form, formulas for solving certain penstock and surge-tank problems, and to derive them from the fundamental principles on which they rest. In the case of ordinary water-hammer, for instance, for which every pipe should be investigated, there is no literature in English except certain incomplete translations from the Italian and Russian. Most of the formulas given in standard books are either limited in their application, incorrect, or contain typographical errors, making them useless. Their results differ by 100 per cent. It is with the object of clearing up such errors that this paper is presented.

Digest of Paper.—The paper may be classified under three headings:

Water-Hammer;
Surge in Surge Tanks; and
Economical Penstock Size.

Under the heading, "Water-Hammer", the ordinary formula for maximum water-hammer is first derived. A simple formula for ordinary cases, when the gate is not closed instantaneously, is then given. The formulas of other writers for this case are also derived, and

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

their limitations and inaccuracies are shown by numerical examples.

Under the heading, "Surge in Surge Tanks", a new formula is given, with a mathematical derivation which it is hoped will be readily understood and followed.

A simple formula for Economical Pipe Size is given for use in water-power developments having long pipes or penstocks.

Conclusions.—Although water-hammer and surge-tank design are common problems to the hydraulic engineer, English literature on these subjects is scattered and incomplete. From constant copying, the formulas have become wrongly applied, and for special cases they are sometimes given as general. Starting with fundamental principles, these problems can be solved by any engineer, and are less complicated than may appear at first sight.

Introduction.

The purpose of this paper is, first, to give in their simplest form for ready reference the best formulas for solving certain surge-tank and penstock problems; and second, for the benefit of engineers who do not believe a formula merely because they see it in print, completely to derive each formula from the fundamental principles on which it rests. All the corresponding formulas which have been found in current literature are then given, and finally numerical examples are worked out, with a comparison of numerical results, using all the different methods. A bibliography of the subject is appended.

Many of these formulas have been published before in standard works, but the writer has found in many cases typographical and other errors that rendered them misleading or incorrect. In most cases no derivation is given, and therefore such errors are difficult to locate.

Formula (B) for ordinary water-hammer, Formula (G)* for surge tanks, and Formula (H) for economical pipe diameter have not been presented before, as far as the writer knows. Moreover, the usual general formulas for water-hammer given in standard works all appear to be more or less limited in their application.

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*Since the foregoing was written, a series of articles on surge tanks, by Professor Prasil, has appeared in *The Canadian Engineer*, in which the method of attack is very much the same.

The writing of this paper was prompted by the fact that, in investigating the safety of a certain pipe design against water-hammer, the formulas given by one authority showed it to be safe, and another formula indicated that the pipe thickness should be doubled. All the best literature on the subject was found to be in Italian or Russian.

On comparing the formulas from all sources, it appeared that one, Formula (*A*), on which all writers agreed, applied up to a certain critical time. Beyond this time another formula applied. At the critical time alone, both formulas held.

On testing the different formulas for this critical time, the results differed widely and were inconsistent with Formula (*A*). Some gave impossible results above the theoretical maximum; with others the results were half as large.

This showed that some of the formulas were fundamentally wrong, and for this reason a rational general formula was sought.

The following is not intended to be an exhaustive treatise on the subject of penstocks and surge tanks. Certain complications, such as governor action, are omitted altogether. The formulas are derived from the maximum values, in which the engineer is most interested, and against which he has to design. Formulas for cases in which only part load is thrown off or on are not given, as the results are less, numerically, and the complication of dealing with two velocities is considerable. In the case of starting or stopping, one of these velocities becomes zero. The mathematics has been simplified as much as possible, and all assumptions have been clearly stated.

The derivations are given, not merely for the value of the resulting formulas, but as a basis for discussion and a starting point from which other special cases can be worked out.

Nomenclature.

The following gives the meaning of every term used in the formulas. All units are given in feet, pounds, and seconds, but any other consistent system will apply equally well.

A = cross-section area of surge tank, in square feet;
a = velocity of vibration along pipe, in feet per second;
b = thickness of pipe walls, in feet;
B = cost of pipe in place, per foot of diameter per foot of length;
C = coefficient in Chezy formula for friction in pipes;

D = inside diameter of pipe, in feet;

E = modulus of elasticity of pipe material in tension, taken as 4 000 000 000 lb. per sq. ft. for steel plate (28 000 000 lb. per sq. in.);

e = 2.718 base of Napierian logarithms;

F = total feet of head lost in pipe between reservoir and standpipe with Q cu. ft. per sec. flowing;

g = acceleration of gravity = 32.2 ft. per sec. per sec.;

h = head due to water-hammer alone (in excess of static head), in feet;

i = income, in dollars per year per foot of head;

K = modulus of elasticity of water in compression, taken as 42 400 000 lb. per sq. ft. (294 000 lb. per sq. in.);

L = length of pipe, in feet;

$$m = \frac{F}{2Q}\sqrt{\frac{A\,g\,P}{L}};$$

$$m' = \frac{m}{\sqrt{1 - m^2}};$$

$$N = \left(\frac{L\,V}{g\,T\,Y}\right)^2;$$

p = percentage return on investment, including profit, depreciation, maintenance, and taxes;

P = cross-sectional area of pipe, in square feet;

Q = steady flow through pipe, in cubic feet per second; in stopping, before gate starts to close; in starting, after gate is open and equilibrium is established;

q = square root of mean square of flow through penstock, in cubic feet per second. (See discussion under Economical Penstock Size.)

S = maximum surge, up or down, in feet, measured, in starting, from reservoir level; and, in stopping, from a distance below this equal to the friction head, F;

s = distance of surge-tank level from starting point at any time, in feet;

T = time of closing of gate, in seconds;

t = time after starting to close or open gate, in seconds;

t_1 = time to reach maximum draw-down, in seconds;

τ = time for a complete cycle of oscillations in the surge tank, in seconds;

u = velocity through gate at any time, in feet per second;

V = velocity of water in pipe, in feet per second corresponding to Q;

v = velocity at any time, in feet per second;

v_0 = velocity near gate at any time, in feet per second;

W = density of water, taken as 62.4 lb. per cu. ft.;

x = distance along pipe, in feet, plus, in the direction from gate to reservoir;

Y = normal static head, in feet;

y = head, in feet, at any point at any time;

y_0 = head, in feet, on gate at any time;

$\psi(t) = \dfrac{\text{area of gate}}{\text{area of pipe}}$, at time, t.

Summary of Formulas.

Maximum Water-Hammer.—$\left(T \text{ less than } \dfrac{2L}{a}\right)$.—The maximum water-hammer which is possible in a pipe will occur with instantaneous closing, or for any closing time less than $\dfrac{2L}{a}$. For this case

$$h = \frac{aV}{g} = \frac{145\,V}{\sqrt{1 + \dfrac{KD}{Eb}}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (A)$$

$$a = \frac{4\,660}{\sqrt{1 + \dfrac{KD}{Eb}}}$$

For steel pipe these become

$$h = \frac{145\,V}{\sqrt{1 + 0.01\,\dfrac{D}{b}}}$$

$$a = \frac{4\,660}{\sqrt{1 + 0.01\,\dfrac{D}{b}}}$$

Ordinary Water-Hammer.—$\left(T \text{ greater than } \dfrac{2L}{a}\right)$.—For ordinary cases

$$h = \frac{LV}{g\left(T - \dfrac{L}{a}\right)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

Allievi gives the following, which is applicable in slow closing, but may be incorrect for quick closing:

$$\frac{h}{Y} = \frac{N}{2} \pm \sqrt{\frac{N^2}{4} + N} \quad \ldots\ldots (C)$$

$$h = \frac{N\,Y}{2} \pm Y\sqrt{\frac{N^2}{4} + N}$$

$$N = \left(\frac{L\,V}{g\,T\,Y}\right)^2$$

Surge in Surge Tanks.—The formulas for the maximum surge in a surge tank, either in starting or stopping the plant, are as follows:

In starting, S is measured from reservoir level; in stopping, it is measured from a distance below this equal to the friction head.

Neglecting friction

$$S = Q\sqrt{\frac{L}{A\,g\,P}} \quad \ldots\ldots (D)$$

A first approximation, allowing for friction, is

$$S = F + Q\sqrt{\frac{L}{A\,g\,P}} = F\left[1 + \frac{1}{2\,m}\right] \quad \ldots\ldots (E)$$

A closer approximation, allowing for friction, is

$$S = F + \frac{Q\sqrt{\frac{L}{A\,g\,P}}}{e^{\frac{\pi}{2}m}} = F\left[1 + \frac{1}{2\,m\,e^{\frac{\pi}{2}m}}\right] \quad \ldots\ldots (F)$$

where $m = \dfrac{F}{2\,Q}\sqrt{\dfrac{A\,g\,P}{L}}$.

The most nearly accurate formula is

$$S = F\left[1 + \frac{1}{2\,me^{m'(\pi - \cos.^{-1} m)}}\right] \quad \ldots\ldots (G)$$

where $m' = \dfrac{m}{\sqrt{1 - m^2}}$.

Economical Penstock Size.—The formula for economical penstock diameter is as follows:

$$D = \sqrt[6]{\frac{3\,250\,q^2\,i}{C^2\,p\,B}} \quad \ldots\ldots (H)$$

Maximum Water-Hammer.

Derivation of Formula.—Formula (*A*) may be derived at once from the fundamental concepts of work and energy.

The maximum pressure in the pipe will occur when the moving water is brought to rest instantaneously. Any section of the pipe in which the water is moving contains a known amount of energy, and, in order to bring this water to rest, all this energy must be dissipated in some way.

With instantaneous closing, the only possible outlet for the energy is:

(1) In compressing the water in the pipe,

(2) In distending the pipe.

Writing the expression for these three forms of energy, and simplifying, Formula (*A*) is obtained in one step.

We assume that all the energy of any section, l, of the pipe goes into distending the pipe and compressing the water in that particular section. The correctness of this assumption may be shown in three ways.

First.—At the gate the first element of water is stopped instantaneously, and it is impossible for it to act on any other section of the pipe than that section; similarly, we can work up the pipe step by step.

Second.—Equation (9*A*), given later under the heading "Ordinary Water-Hammer", shows that the pressure is propagated up the pipe as a wave without loss of intensity, and, therefore, all parts of the pipe attain the same maximum pressure.

Third.—By experiment with very fast closing of gates, it has been found that the pressure rise is constant at all parts of a given pipe and for practically any length of pipe.

The energy of the moving water in a section of pipe of any length, l, is

$$\text{weight} \times \frac{\text{velocity}^2}{2g}$$

$$= \frac{W \pi D^2 l}{4} \times \frac{V^2}{2g}$$

$$= \frac{W \pi D^2 l V^2}{8g} \text{ foot-pounds} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The maximum pressure tending to compress the water in the pipe is

$$Wh \frac{\pi D^2}{4} \text{ (in pounds)},$$

and, as this starts at zero and increases uniformly, the average pressure is half as great.

The distance (measured along the pipe) which l feet of water will be compressed is

$$\frac{Wh\,l}{K} \text{ feet.}$$

The work done in compressing l feet of the water, therefore, is

$$\frac{Wh\ \pi D^2}{4} \times \frac{1}{2} \times \frac{Wh\,l}{K}$$

$$= \frac{W^2h^2\,\pi\,l\,D^2}{8\,K} \text{ foot-pounds.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The maximum tension tending to burst a section of pipe of length, l, is

$$\frac{Wh\,l\,D}{2} \text{ pounds, maximum pressure.}$$

As this starts at zero and increases uniformly, the average tension is half of this.

The distance which the ring of steel forming the pipe will stretch is

$$\frac{Wh\,D\,l\,\pi\,D}{2\,b\,E\,l} = \frac{Wh\,\pi\,D^2}{2\,b\,E} \text{ feet.}$$

The work done in stretching this steel, therefore, is

$$\frac{Wh\,D\,l}{2} \times \frac{1}{2} \times \frac{Wh\,\pi\,D^2}{2\,b\,E}$$

$$= \frac{W^2h^2\,\pi\,l\,D^3}{8\,b\,E} \text{ foot-pounds} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Setting the expression (1) for the energy in the pipe equal to the two expressions (2) and (3) for the work done by the water in coming to rest, we get

$$\frac{W\,\pi\,l\,D^2\,V^2}{8\,g} = \frac{W^2h^2\,\pi\,l\,D^2}{8\,K} + \frac{W^2h^2\,\pi\,l\,D^3}{8\,b\,E},$$

and simplifying

$$h^2 = \frac{V^2}{W\,g\left(\frac{1}{K} + \frac{D}{b\,E}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$h = \sqrt{\frac{K}{W\,g}\,\frac{V^2}{\left(1 + \frac{K\,D}{E\,b}\right)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Inserting the numerical value of $\sqrt{\frac{K}{W\,g}}$, this becomes

$$h = \frac{145\ V}{\sqrt{1 + \frac{K\,D}{E\,b}}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

Inserting values for steel pipe:

$$K = 4.2 \times 10^7$$
$$E = 4.0 \times 10^9$$

this becomes

$$h = \frac{145\ V}{\sqrt{1 + 0.01\,\frac{D}{b}}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Other Derivations.—Formula (*A*) has been derived by several other methods. One is as follows:

The velocity of a vibration along a pipe is shown to be

$$a = \sqrt{\frac{g}{W\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} = \frac{4\,660}{\sqrt{1 + \frac{K\,D}{E\,b}}} \qquad \ldots\ldots\ldots\ldots\ldots (12)$$

(See Formula (12) under the heading "Ordinary Water-Hammer").

If the gate is closed instantly, the pressure will rise immediately to h at the gate, but it will take a time, $\frac{L}{a}$, before this rise has reached the upper end of the pipe, and before the last layer of water is brought to rest.

The principle of impulse and momentum is that the impulse (or force multiplied by time) of all forces acting on a collection of particles in any time is equal to the gain of momentum of the particles in that time (mass multiplied by velocity). In this case

$$-Wh\,P\,\frac{L}{a} = -\frac{W\,P\,L}{g}\,V$$

$$h = \frac{V}{g}\,a \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Inserting the value of a from Formula (12) we get, as before,

$$h = \frac{4\,660\ V}{g\sqrt{1 + \frac{K\,D}{E\,b}}} = \frac{145\ V}{\sqrt{1 + \frac{K\,D}{E\,b}}} \qquad \ldots\ldots\ldots\ldots\ldots (A)$$

One point should be emphasized, namely, that as all the energy in the water is used in producing this maximum h, it is impossible to get a greater pressure than this. Therefore, any formula yielding greater results must be incorrect, as it violates the law of the conservation of energy.

What appears to the writer to be an incorrect use of the formula is given in one of the articles on this subject. This inserts the ordinary value of E_w for wood, and applies the formula to wood stave pipes.

If the staves are initially in compression, the term, Eb, should be replaced by $E_w\, b + E_s\, \phi$,

where E_w is the modulus of elasticity of the wood,
E_s " " " " " " " steel,
and ϕ " " cross-section of steel bands per unit of length of pipe.

If the staves are not initially in compression, the $E_w\, b$ drops out altogether. In any case, the sinking of the steel bands into the wood is neglected.

Ordinary Water-Hammer.

Derivation of Formula.—The maximum value of the water-hammer given by Formula (A) cannot occur if the reflected waves from the reservoir reach the gate before it is entirely closed. These waves return in what may be called the critical time, equal to $\frac{2\,L}{a}$, and, for cases where T is greater than $\frac{2\,L}{a}$, the maximum value will have to be less than that derived from Formula (A).

The rise of pressure with gradual closing is more difficult to determine than with instantaneous closing, but, as almost all cases come under this head, the problem is worthy of careful analysis.

When the gate is closed a slight amount, a wave is started up the pipe, and the pressure at the gate rises at once. If the gate is closed no farther, this pressure will continue constant until $T = \frac{2\,L}{a}$, at which time the reflected wave from the reservoir reaches the gate, and the pressure falls to normal. The pressure then falls below normal by an amount which would be identical with the above rise, if friction, etc., did not tend to damp out the vibrations. It then rises again, and repeats the cycle until entirely dissipated by friction.

If the gate continues closing, however, a new wave and corresponding rise of pressure will be started for each increment of the gate motion, and as long as the gate moves the pressure will continue to rise until the reflected waves interfere. It is assumed that the gate is closed in such a way that, in the absence of reflected waves,

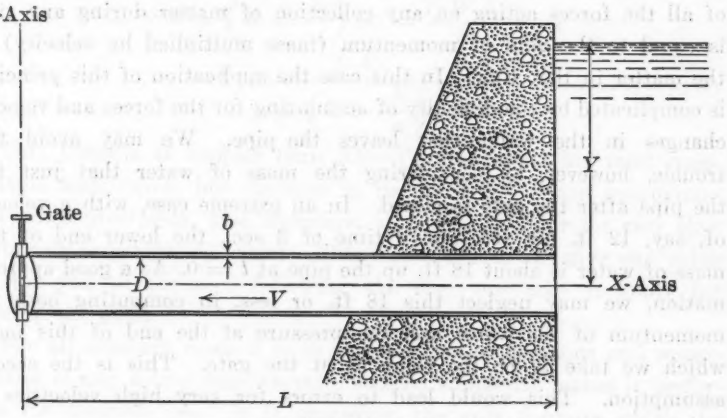

FIG. 1.

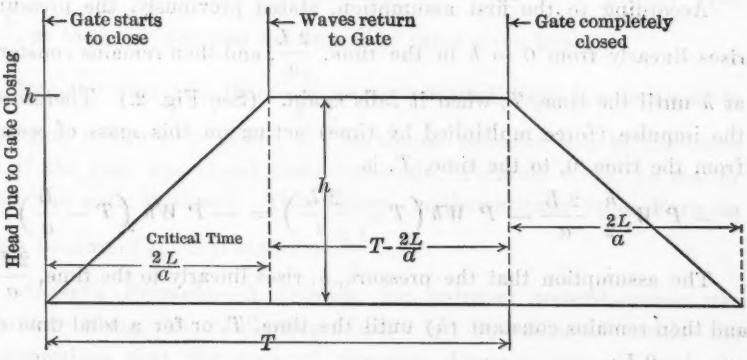

FIG. 2.

the pressure will rise at a constant rate. When the waves come back, the rate at which they are diminishing the pressure is the same as that at which the gate is increasing it. (See Fig. 2.) The pressure, therefore, will become constant until the gate is stopped, when it will begin to fall again, reaching normal after a time, $\frac{2L}{a}$. This is the

first assumption, and is very near to the truth, as experiments have shown.

In attempting to find a general formula, the principle of impulse and momentum was used. In its ordinary form, this principle is that the resultant impulse (force multiplied by time) or time integral of all the forces acting on any collection of matter during any time is equal to the gain of momentum (mass multiplied by velocity) of the matter in that time. In this case the application of this principle is complicated by the difficulty of accounting for the forces and velocity changes in the water that leaves the pipe. We may avoid this trouble, however, by considering the mass of water that just fills the pipe after the gate is closed. In an extreme case, with a velocity of, say, 12 ft. per sec. and a time of 3 sec., the lower end of this mass of water is about 18 ft. up the pipe at $t = 0$. As a good approximation, we may neglect this 18 ft. or less, in computing both the momentum of the water and the pressure at the end of this mass, which we take to be the same as at the gate. This is the second assumption. This would lead to errors for very high velocities or extremely slow closing, but not for ordinary cases.

According to the first assumption, stated previously, the pressure rises linearly from 0 to h in the time, $\frac{2L}{a}$, and then remains constant at h until the time, T, when it falls again. (See Fig. 2.) Therefore, the impulse (force multiplied by time) acting on this mass of water from the time, 0, to the time, T, is

$$-PW\frac{h}{2}\frac{2L}{a} - PWh\left(T - \frac{2L}{a}\right) = -PWh\left(T - \frac{L}{a}\right).$$

The assumption that the pressure, h, rises linearly to the time, $\frac{2L}{a}$, and then remains constant (h) until the time, T, or for a total time of $\left(T - \frac{2L}{a}\right)$, restricts the application of the formula to times greater than $\frac{2L}{a}$.

The velocity of the part in the pipe changes from V to 0*, and its

*A rigorous analysis from Allievi (given later) shows that, with the above assumptions, the velocity of the water at all points of the pipe is exactly 0 at $t = T$. This can be seen from the fact that, first, as the gate is closed, there is no velocity due to water flowing through the pipe; second, as the waves running up the pipe are exactly equal in intensity to the reflected waves coming down the pipe, there is no motion due to this cause.

mass, according to the second assumption, is $\frac{P\ W\ L}{g}$. The gain of momentum (mass multiplied by velocity), therefore, is $-\frac{P\ W\ L}{g}\ V$.

The equation of impulse and momentum, therefore, is

$$-P\ W\ h\left(T-\frac{L}{a}\right)=-\frac{P\ W\ L}{g}\ V, \text{ or}$$

$$h=\frac{L\ V}{g\left(T-\frac{L}{a}\right)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

This formula is simpler than Allievi's, and omits his most questionable assumption, but contains in its stead the second approximation noted previously, and, therefore, is subject to error for very slow closing or very high velocities. This formula does not give any value for the fall of pressure following the rise, nor does it apply for opening the gates.

For all ordinary cases it is as close as the assumptions and uncertainties of any actual case warrant, and its simplicity makes it preferable for common use. It is, moreover, the only formula which the writer has ever seen that is good for all values of T greater than $\frac{2\ L}{a}$, as one may see from the numerical cases given later.

For any time less than $\frac{2\ L}{a}$, as no reflected wave has returned to the gate before it is closed, the maximum pressure will be the same as if the gate was closed instantaneously (Formula (A)), no matter how the gate is closed. This follows mathematically from Formula (14) because $f\ (t) = 0$ until $t = \frac{2\ L}{a}$.

Allievi's Formula.—A formula for ordinary water-hammer was derived along the following lines by L. Allievi,* with the assumptions that the area of the gate decreases linearly and the pipe velocity decreases at a constant rate throughout its closure.† It also assumes that T is not less than $\frac{2\ L}{a}$ and is good for such values of T only. The assumption of a linear rise of pressure, which is involved in that of a linear decrease of velocity, is confirmed by actual

* *Annali* della Societa degli Ingegneri, Vol. XVII, Rome, 1902.

† These two assumptions do not seem to be exactly consistent. The second alone is assumed in deriving Formula (B), and no assumption about the gate area is made there.

experiments under different conditions and different gate motions, and the formula itself has been checked for ordinary closing of gates where T is considerably greater than $\frac{2L}{a}$.

We assume the X-axis to be parallel with the pipe along the center line, starting from the gate, and plus in the direction of the reservoir. The sense of v is opposed to this being plus in the direction from the reservoir to the gate. The pipe is assumed to be horizontal, of constant diameter and thickness, and under pressure, but the formulas hold for inclined pipes as well. (See Fig. 1.)

The first fundamental equation is that the acceleration of any thin element of water of area, P, and thickness, $d\,x$, is caused by a difference of head on its two sides.

Therefore, as mass multiplied by acceleration equals force,

$$W\,P\left(y + \frac{\delta\,y}{\delta\,x}\,d\,x\right) - W\,P\,y = \frac{P\,W\,d\,x}{g}\,\frac{\delta\,v}{\delta\,t} \quad \ldots\ldots(8)$$

$$\text{Simplifying, } \frac{\delta\,v}{\delta\,t} = g\,\frac{\delta\,y^*}{\delta\,x} \quad \ldots\ldots(9)$$

which is the first fundamental equation.

The second equation results from the fact that there is a difference of velocity at any time at different points in the pipe in consequence of the deformations due to the rise in head, $d\,y$, which causes

First.—A compression of the water;

Second.—An enlargement of section due to stretching of the steel ring of the pipe which must be filled with water.

The first part of this is

$$\frac{d\,x}{K}\,W\,\frac{\delta\,y}{\delta\,t}\,d\,t \quad \ldots\ldots(10)$$

which is the distance along the pipe in which the water is compressed.

The lengthening of the pipe ring due to y is

$$\frac{W\,y}{E} \times \frac{D}{2\,b}\,\pi\,D = \frac{\pi\,D^2\,W\,y}{2\,E\,b} \text{ (See Formula (3).)}$$

The corresponding lengthening of diameter is

$$D_1 = \frac{D^2}{2\,E\,b}\,W\,y$$

*For inclined pipes, this becomes $\frac{\delta\,v}{\delta\,t} = g\,\frac{\delta\,y}{\delta\,x} + g\,\sin.\,\alpha$, where α is the angle of inclination with the horizontal.

In a time, $d\,t$, this variation due to $d\,y$ is

$$d\,D_1 = \frac{D^2\,W}{2\,E\,b}\,\frac{\delta\,y}{\delta\,t}\,d\,t.$$

The corresponding enlargement of volume of an element of the pipe of length, $d\,x$, and area, $\frac{\pi\,D^2}{4}$, is

$$\frac{2\,\pi\,D\,d\,D_1\,\,d\,x}{4}\,; \qquad (d\,D_1 = d\,D),$$

or, inserting the above value for $d\,D_1$,

$$\frac{\pi\,D^3\,W}{4\,E\,b}\,\frac{\delta\,y}{\delta\,t}\,d\,t\,d\,x$$

Dividing the volume by the area to get the difference in the distances along the pipe which the ends of the column have moved in taking up this new volume

$$\frac{D\,W}{E\,b}\,\frac{\delta\,y}{\delta\,t}\,dt\,dx \qquad \ldots\ldots(11)$$

which is the second part required.

The difference in velocities between any two points of the pipe is due to the sum of these two, or

$$\left(v + \frac{\delta\,v}{\delta\,x}\,dx\right)\,dt - v\,dt \;=\; \text{Formulas (10)} + (11),$$

or, inserting the values of Formulas (10) and (11) and simplifying,

$$\frac{\delta\,v}{\delta\,x} = \left(\frac{1}{K} + \frac{D}{E\,b}\right)\,W\,\frac{\delta\,y}{\delta\,t},$$

which is the second equation necessary.

Let

$$\frac{1}{a^2} = \frac{W}{g}\left(\frac{1}{K} + \frac{D}{E\,b}\right);^*$$

$$a = \sqrt{\frac{g}{W\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} \qquad \ldots\ldots(12)$$

substituting this,

$$\frac{\delta\,v}{\delta\,x} = \frac{g}{a^2}\,\frac{\delta\,y}{\delta\,t} \qquad \ldots\ldots(13)$$

*a is really the velocity of the wave along the pipe, as is seen from Formulas (9A) or (14).

By elimination of v in Equations (9) and (13) we get the common wave equation,

$$\frac{\delta^2 y}{\delta x^2} = \frac{1}{a^2}\frac{\delta^2 y}{\delta t^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9A)$$

likewise,

$$\frac{\delta^2 v}{\delta x^2} = \frac{1}{a^2}\frac{\delta^2 v}{\delta t^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13A)$$

A general solution of Equations (9) and (13), or (9A) and (13A), is

$$y = Y + F\left(t - \frac{x}{a}\right) - f\left(t + \frac{x}{a}\right)^* \quad \ldots\ldots\ldots (14)$$

$$v = V - \frac{g}{a}\left[F\left(t - \frac{x}{a}\right) + f\left(t + \frac{x}{a}\right)\right]$$

these being general expressions for waves traveling in both directions with velocity, a.

Introducing in the first part of Equation (14) the fact that when $x = L$, $y = Y$, for all values of t, we get

$$f\left(t + \frac{L}{a}\right) = F\left(t - \frac{L}{a}\right) \quad \ldots\ldots\ldots\ldots\ldots (15)$$

To get rid of the unknown function, $f\left(t + \frac{x}{a}\right)$, we may express it in terms of F. To do this, choose t_1 so that

$$\left.\begin{aligned} t_1 + \frac{L}{a} &= t + \frac{x}{a} \\ t_1 - \frac{L}{a} &= t + \frac{x}{a} - \frac{2L}{a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (16)$$

Therefore, from Equations (15) and (16),

$$f\left(t + \frac{x}{a}\right) = f\left(t_1 + \frac{L}{a}\right) = F\left(t_1 - \frac{L}{a}\right) = F\left(t + \frac{x}{a} - \frac{2L}{a}\right). \quad (17)$$

Formulas (17) and (14) give

$$\left.\begin{aligned} y &= Y + F\left(t - \frac{x}{a}\right) - F\left(t + \frac{x}{a} - \frac{2L}{a}\right) \\ v &= V - \frac{g}{a}\left[F\left(t - \frac{x}{a}\right) + F\left(t + \frac{x}{a} - \frac{2L}{a}\right)\right] \end{aligned}\right\} \quad \ldots\ldots (18)$$

By Taylor's theorem

$$\left.\begin{aligned} F\left(t - \frac{x}{a}\right) &= F(t) - \frac{x}{a}F'(t) + \Sigma_1 \\ F\left(t + \frac{x}{a} - \frac{2L}{a}\right) &= F(t) - \frac{2L - x}{a}F'(t) + \Sigma_2 \end{aligned}\right\} \quad \ldots\ldots (19)$$

*For inclined pipes this would have a term, $-x \sin. \alpha$, which would disappear at $x = 0$, the gate.

in which Σ_1 and Σ_2 represent terms containing derivatives of $F(t)$ of higher order. If we now suppose Σ_1 and Σ_2 to be negligible,* substituting Equation (19) in Equation (18), we get

$$y = Y + 2\frac{L-x}{a}F'(t). \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

$$v = V - \frac{2g}{a}\left[F(t) - \frac{L}{a}F'(t)\right].$$

We assume the rise of pressure to be at a constant rate until the wave returns, that is

$$F'(t) = \text{constant}.$$

Thus, by Equation (20), as y_0 and v_0 are the values of y and v, respectively, at $x = 0$ (the gate),

$$\frac{dy_0}{dt} = 0 \text{ and } \frac{d^2 v_0}{dt^2} = 0$$

at any time after the reflected wave returns. As the equation of flow (neglecting friction) is

$$u^2 = \frac{v_0^2}{[\psi(t)]^2} = 2gy_0$$

and as $\frac{dy_0}{dt} = 0$ (the assumption of linearity),

$$\frac{dv_0}{dt} = \psi'(t)\sqrt{2gy_0}.$$

Differentiating the second part of Equation (20) for $x = 0$, $v = v_0$, we get

$$\frac{dv_0}{dt} = -\frac{2g}{a}F'(t).$$

Equating these two we get a value of $F'(t)$ which, substituted in the first part of Equation (20), gives a second degree equation in y_0 from which h, or $y_0 - Y$, can be obtained. Now, remembering that from linearity, $\psi'(t) = -\frac{V}{T\sqrt{2gY}}$,† and calling $N = \left(\frac{LV}{gTY}\right)^2$

$$\frac{y_0^2}{Y^2} - 2\frac{y_0}{Y}\left[1 + \frac{1}{2}\left(\frac{LV}{gTY}\right)^2\right] + 1 = 0. \quad \ldots\ldots\ldots (21)$$

* Σ_1 is not negligible if $\frac{x}{a} > t$, as the wave has not yet arrived at the point. Likewise, Σ_2 is not negligible if $\frac{2L-x}{a} > t$, as the reflected wave has not yet got back there. Therefore, the following formulas are incorrect if the time of closing is less than $\frac{2L}{a}$. Mathematically, this follows, because F is defined by different algebraic expressions for positive and negative values of its argument. Thus, a Taylor's series for positive values will not hold for negative values.

† This brings in the assumption of linear gate closing, which is used here only. This is probably inconsistent with the other assumption of linear rise of pressure.

Solving this, and putting $h = y_0 - Y$, since $t > \frac{2L}{a}$,

$$\left.\begin{aligned} \frac{h}{Y} &= \frac{N}{2} \pm \sqrt{\frac{N^2}{4} + N} \\ h &= \frac{NY}{2} \pm Y\sqrt{\frac{N^2}{4} + N} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (C)$$

As a matter of fact, this formula becomes inaccurate as it approaches $\frac{2L}{a}$. This discrepancy must be due to some error in the assumptions. The assumption that ψ (t) and F (t) are both linear functions seems to be most open to question.

Experiments have shown that F (t) (the rise of pressure) is often linear. Therefore, the other assumption, namely, that ψ (t) (rate of closing of gate), is at the same time linear is probably at fault. This latter assumption is not used in obtaining Formula (B).

The minus sign in Formula (C) is supposed to apply in opening the gates, and the drop of pressure, according to the formula, can never exceed Y, but approaches it as a limit.

Surge in Surge Tanks.

The problem of ordinary surge-tank design is simple, as far as the theory and fundamental assumptions are concerned. In starting up, the water is at rest in the entire system. Suddenly the load is thrown on, and, assuming that the full load flow is immediately drawn from the surge tank, a known quantity of water is drawn out, and the quantity of water coming in depends on the difference in head between the surge tank and the reservoir.

Neglecting friction, this problem is of the simplest, and can be derived at once from work and energy. The answer is the same, either for starting or stopping the plant.

When friction is considered, the fundamental equations are still simple to set up, but the solution becomes difficult, even with the aid of the higher mathematics. The following solution is offered by Dr. D. L. Webster, of the Physics Department of Harvard University:

To determine the motion of the water in the pipe, consider the work done on it and the kinetic energy it acquires in moving an

infinitesimal distance, dx. The amounts of work done by various forces are as follows:

(*a*) By gravity: $(W\,P\,dx)\,s$

(*b*) By friction: $(W\,P\,dx)\,\left(-f\,\frac{v^2}{V^2}\right)$,

f being the head loss by friction alone at the limiting velocity, V, and, therefore, $f\,\frac{v^2}{V^2}$ being the loss at the velocity, v. (Friction varies as velocity squared.)

The kinetic energy acquired by the water in the pipe at the same time is $\frac{W\,P\,L}{g}\,d\left(\frac{v^2}{2}\right) = \frac{W\,P\,L}{g}\,v\,dv$; and the energy acquired by the water entering the pipe is $W\,\frac{P\,dx}{g}\,\frac{v^2}{2}$.

Equating the work done to the energy acquired, we find

$$\left[s - f\,\frac{v^2}{V^2} - \frac{v^2}{2\,g}\right]dx = \frac{L}{g}\,v\,dv \quad \ldots\ldots\ldots\ldots\ldots (21)$$

or, as $dx = v\,dt$,

$$\frac{d\,v}{dt} = \frac{g}{L}\left[s - \left(\frac{f}{V^2} + \frac{1}{2\,g}\right)v^2\right] \quad \ldots\ldots\ldots\ldots\ldots (22)$$

This equation may be further simplified by noticing that, in a steady condition, s becomes the final head loss, F, by friction and entrance to the pipe together, and $\frac{d\,v}{dt}$ becomes zero and $v = V$, whence

$$F = f + \frac{V^2}{2\,g} \quad \ldots\ldots\ldots\ldots\ldots (23)$$

and

$$\frac{d\,v}{dt} = \frac{g}{L}\left[s - F\,\frac{v^2}{V^2}\right] \quad \ldots\ldots\ldots\ldots\ldots (24)$$

In this equation, v and s are both variables, dependent on t, so that, as s is the one in which we are interested, it would be well to express the equation in terms of t, s, $\frac{d\,s}{dt}$, and $\frac{d^2\,s}{dt^2}$. This we may do by using the equation

$$A\,s = Q\,t - P\,x \quad \ldots\ldots\ldots\ldots\ldots (25)$$

derived by considering the volumes leaving and entering the standpipe. By differentiating Equation (25), we may obtain

$$A\,\frac{d\,s}{dt} = Q - P\,v, \quad A\,\frac{d^2\,s}{dt^2} = -\,P\,\frac{d\,v}{dt} \quad \ldots\ldots\ldots (26)$$

Therefore, substituting in Equation (24),

$$\frac{d^2 s}{dt^2} - \frac{g F}{P L A V^2}\left(Q - A\frac{d s}{dt}\right)^2 + \frac{g P}{L A} s = 0 \ldots\ldots (27)$$

As it is impossible to integrate this equation, we must make some approximation, preferably one that will make the maximum drawdown greater, rather than less, than its real value, and thus be on the safe side. Such an approximation may be obtained by considering the head loss by friction and entrance proportional to the velocity, rather than to its square, having the actual value, F, when $v = V$, and therefore, a greater value for all smaller velocities.

Thus we may replace $F\frac{v^2}{V^2}$ in Equation (24) by $F\frac{v}{V}$ and replace Equation (27) by

$$\frac{d^2 s}{dt^2} + \frac{g F}{L V}\frac{d s}{dt} + \frac{g P s}{L A} = \frac{g F Q}{L A V} = \frac{g P F}{L A} \ldots\ldots (28)$$

When a steady condition is reached, with $v = V$, $\frac{d s}{dt}$ and $\frac{d^2 s}{dt^2}$ will be zero, and we shall have s equal to F, as we have seen above, or as we can derive from Equation (28), remembering that $\frac{Q}{P} = V$. Letting $s' = s - F$, $2k = \frac{g F}{L V}$, and $n^2 = \frac{g P}{L A}$, Equation (28) becomes

$$\frac{d^2 s'}{dt^2} + 2k\frac{d s'}{dt} + n^2 s' = 0 \ldots\ldots (29)$$

This equation is the common form for damped harmonic vibrations, and is treated at length in many text books on the calculus, such as Osgood's, where the general solution is shown to be of the form

$$s' = C_1 e^{-kt} \cos.\ n' t + C_2 e^{-kt} \sin.\ n' t \ldots\ldots (30)$$

where $n' = \sqrt{n^2 - k^2}$, and C_1 and C_2 are arbitrary constants to be determined from the initial conditions of the problem.

For the case where full power is turned on, with the water in the conduit initially at rest, we have when

$$t = 0, s' = -F \text{ and } \frac{d s'}{dt} = \frac{Q}{A} \ldots\ldots (31)$$

From these we may at once obtain

$$C_1 = -F, \text{ and } -C_1 k + C_2 n' = \frac{Q}{A} \ldots\ldots (32)$$

or

$$s = F\left[1 - e^{-kt}\left(\cos.\ n' t + \frac{k}{n'} \sin.\ n' t\right)\right] + \frac{Q}{A n'} e^{-kt} \sin.\ n' t \ldots\ldots (33)$$

To find the maximum draw-down, we have now merely to differentiate this and set the derivative equal to zero.

Thus we find

$$\frac{ds}{dt} = e^{-kt}\left[F\left\{ k\left(\cos.\ n'\,t + \frac{k}{n'}\sin.\ n'\,t\right) + n'\sin.\ n'\,t - k\cos.\ n'\,t\right\} + \frac{Q}{A}\left(\cos.\ n'\,t - \frac{k}{n'}\sin.\ n'\,t\right)\right]$$

$$= e^{-kt}\left[F\frac{n^2}{n'}\sin.\ n'\,t + \frac{Q}{A}\left(\cos.\ n'\,t - \frac{k}{n'}\sin.\ n'\,t\right)\right] = 0. \quad (34)$$

This expression equals zero when

$$\tan.\ n'\,t = -\frac{n'}{\frac{FA}{Q}n^2 - k} = \frac{-n'}{k},\quad \cos.\ n'\,t = -\frac{k}{n},$$

$$\sin.\ n'\,t = +\frac{n'}{n}$$

Let t_1 = this value of t, when s is at its maximum, S.

From Equation (33) at time, t_1, $s = S$, where

$$S = F\left\{1 - e^{-kt_1}\left(-\frac{k}{n} + \frac{k}{n}\right)\right\} + \frac{Q}{An'}e^{-kt_1}\frac{n'}{n},$$

or,
$$S = F + \frac{Q}{An}e^{-kt_1} = F + Q\sqrt{\frac{L}{gAP}}e^{-kt_1}. \quad (35)$$

This can be put in simpler form, for

$$kt_1 = \frac{k}{n'}\left(\pi - \cos.^{-1}\frac{k}{n}\right) \quad (36)$$

and

$$\frac{k}{n} = \frac{F}{2Q}\sqrt{\frac{AgP}{L}} = m, \text{ and } \frac{k}{n'} = \frac{m}{\sqrt{1-m^2}} = m'$$

Therefore, $k\,t_1 = m'\,(\pi - \cos.^{-1} m)$, and Equation (35) becomes

$$S = F + \frac{Q\sqrt{\frac{L}{gAP}}}{e^{m'(\pi - \cos.^{-1} m)}} = F\left[1 + \frac{1}{2me^{m'(\pi - \cos.^{-1} m)}}\right]. \quad (G)$$

This is the best approximation obtainable by this method.

The time required for a full cycle is $\tau = \frac{2\pi}{n'}$ (36A)

where $n' = \sqrt{n^2 - k^2} = n\sqrt{1-m^2} = \sqrt{\frac{gP}{LA}}\sqrt{1-m^2}$.

Thus $\tau = 2\pi\sqrt{\frac{L\,A}{g\,P} + \frac{F^2\,A^2}{4\,V^2\,P^2}}$, approximately.

The time from starting to maximum draw-down is, from Equation (36)

$$t_1 = \frac{1}{n'}(\pi - \cos.^{-1} m) > \frac{\tau}{4}$$

A second best approximation for S may be obtained, as $\frac{k}{n}$ is small, by taking $n' = n$ and

$$\cos.^{-1}\frac{k}{n} = \frac{\pi}{2} \text{ and } k\,t_1 = \frac{\pi\,k}{2\,n} \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

This gives, for this rougher approximation,

$$S = F + V\sqrt{\frac{L\,P}{g\,A}}\,e^{-\frac{\pi\,m}{2}} = F + \frac{Q\sqrt{\frac{L}{A\,g\,P}}}{e^{\frac{\pi\,m}{2}}} = F\left[1 + \frac{1}{2\,m\,e^{\frac{\pi\,m}{2}}}\right] \ldots\ldots (F)$$

A very rough approximation obtained from Equation (G), neglecting the effect of friction in damping the vibrations, though not in determining the final position, is

$$S = F + Q\sqrt{\frac{L}{g\,A\,P}} = F\left[1 + \frac{1}{2\,m}\right] \ldots\ldots\ldots\ldots (E)$$

A point to be noticed about these approximations is that the best, expressed by Equation (G), should give an answer slightly above the true one, the next best, Equation (F), somewhat higher, and the third best, Equation (E), the highest of all.

Although the most important quantity in the design of a surge tank is the maximum draw-down when throwing on the full load, with the water in the conduit initially at rest, the rise when the full power is suddenly shut off is also important, and may be found as a by-product of the foregoing analysis.

For this problem, the principal change in the analysis is the dropping of the term, Qt, in Equation (25), and the terms, such as the right hand side of Equation (28), that result from it. At the same time, the initial conditions must be changed to

$$s = +F \text{ and } \frac{ds}{dt} = -\frac{Q}{A} \ldots\ldots\ldots\ldots\ldots (31A)$$

at $t = 0$; so that s is determined exactly as s' was in the previous work, but with the sign reversed. The new algebraic minimum value

of s, therefore, will be equal to minus the above maximum of s', making the maximum rise above the position of static equilibrium,

$$Q\sqrt{\frac{L}{g\,A\,P}}\,e^{-k\,t_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

exactly as in Equation (35).

Thus it appears that the rise of the water from its position of kinetic equilibrium is exactly like its previous drop from its position of static equilibrium, so that the maximum height above the static position is less than the maximum draw-down from it by the distance, F, between the equilibrium positions.

Neglecting friction entirely, we get from Equation (E)

$$S = V\sqrt{\frac{L\,P}{g\,A}} = Q\sqrt{\frac{L}{g\,A\,P}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (D)$$

which is true for either starting or stopping.

This simple form can be checked at once by work and energy, since the work done in the surge tank is $\frac{1}{2}\,S^2\,W\,A$, and the work done in starting or stopping the water equals its kinetic energy, or $\frac{W\,P\,L\,V^2}{2\,g}$; equating these

$$\frac{1}{2}\,W\,A\,S^2 = \frac{1}{2}\,W\,\frac{P\,L\,V^2}{g}$$

$$S = V\sqrt{\frac{L\,P}{g\,A}} = Q\sqrt{\frac{L}{g\,A\,P}} \text{ as before} \quad \ldots\ldots\ldots\ldots (D)$$

Economical Size of Penstock.

In developments involving the use of long flow lines, or pipes under a high head, the cost of the pipe line is sometimes the largest part of the total cost. For this reason its most economical size becomes a matter of the greatest importance.

The following formula was first derived for a rough check on the independent calculations of several engineers in determining the best size for a certain pipe. The result of the somewhat academic mathematical treatment was so simple that it led the writer to suspect its practical value. It was not until after it had been checked repeatedly by other methods, starting with quite different assumptions, that it seemed sufficiently useful to present for general use.

One point especially should be emphasized, namely, that a wide variation in the assumptions will lead to a comparatively slight variation in result, and the first approximation in which the assumptions may be fairly rough will generally yield a result very close to the final figure. For instance, a variation of 100% in any one of the assumptions will make a difference of 1 ft., only, in a 10-ft. pipe.

Derivation.—By the Chezy formula, the head lost in a pipe is

$$\frac{L\,V^2}{C^2\,R}\text{ feet}\qquad(40)$$

where $$R = \frac{D}{4}.$$

The income lost from this loss of head is

$$\frac{L\,V^2}{C^2\,R}\,i\text{ dollars per year}\qquad(41)$$

This is clear loss, as the operating and maintenance expenses of the plant are assumed to be fixed.

The other factor which must be considered is the annual fixed charge on the pipe. This is

$$\frac{p}{100}\,B\,L\,D\text{ dollars per year}\qquad(42)$$

It is clear that the sum of these two factors is the total yearly loss, and a pipe having a diameter which will make this sum a minimum will be most economical. Adding these and inserting the value $R = \frac{D}{4}$ and $V = \frac{q}{\frac{\pi D^2}{4}}$, the yearly loss is

$$\frac{16 \times 4 \times L\,q^2\,i}{C^2\,D^4\,\pi^2\,D} + \frac{p\,B\,L\,D}{100}\qquad(43)$$

or simplifying,

$$\frac{6.5\,L\,q^2\,i}{C^2\,D^5} + \frac{p\,B\,L\,D}{100}\qquad(44)$$

In order to get the minimum value of this expression, with variation of D, it is merely necessary to differentiate once and set equal to zero.

$$0 = \frac{-5 \times 6.5 \times L\,q^2\,i}{C^2\,D^6} + \frac{p\,B\,L}{100}\qquad(45)$$

$$\frac{3\,250\,L\,q^2\,i}{C^2\,D^6} = p\,B\,L\qquad(46)$$

$$D = \sqrt[6]{\frac{3\,250\,q^2\,i}{C^2\,p\,B}}\qquad(H)$$

Assumptions.—The foregoing formula depends for its accuracy on the exactness of the factors which enter into it, and although these may be varied considerably without materially affecting the results, nevertheless certain principles should be carefully noted.

Flow Through Penstock.—The most difficult factor to select in Equation (*H*) is q, the flow through the penstock.

In a plant with a single pipe line running at full load all its operating time, and under a constant head, the full load flow, Q, would of course be used. In other plants the probable operating conditions must be known and q selected accordingly. Certain generalities, however, apply to all cases.

Where there is more than one penstock, it is assumed that each one will carry a proportional part of the load throughout the year, even though one of them may seldom be used.

If a typical daily load curve can be assumed the flow at each hour of the day can be obtained and the square root of the average square may be used for q. The average is not used, for the loss of head varies as the velocity squared, and not as the velocity. The only time considered should be that in which the plant is in active operation, for when it is closed down, or when the wheels are merely turning over, there is no income being earned and the loss of head, if any, makes no difference.

As a matter of interest, it may be said that in several cases which were studied, with load factors varying from 10 to 40%, the value of q chosen was about 80% of the full load flow.

Income per Foot of Head.—The income per foot of head is the total gross income of the plant divided by the average net head and the number of penstocks. It was calculated that a 10 000-kw. plant with four penstocks under a 100-ft. head would earn $160 000 gross per year, or $40 000 per unit. This gives $400 per year per foot of head for each penstock, but for six-tenths of the year there was an excess of water and a loss of 2 or 3 ft. in the penstock could be offset by opening the gate and using more water at a sacrifice of efficiency. Therefore, only $160 a year should be used for i in this case, as it is only during low-water periods that a larger penstock will earn additional income.

Percentage of Investment.—The percentage return on the investment which the initial cost of the penstocks is expected to give covers

fixed charges, depreciation, taxes, profit, and in fact everything. The profit should be the same as that which the total plant investment is expected to yield.

Cost of Penstock.—The formula assumes that the cost of the penstock in place varies as the diameter. This assumption is very close to the truth, within the narrow limits in which the penstock size may vary. For this variation the commercial thickness of plates or staves is the determining factor in pipe cost. The quantities of excavation and back-fill vary with the diameter squared, but it would not be proper to assume that the cost varied correspondingly, as the equipment, fixed charges, etc., form so large a part that a small percentage change in quantities would vary the cost more nearly as the diameter than as the square of the diameter.

For the first trial a penstock size must be assumed before the cost can be estimated or the formula used. If the economical size differs greatly from this, a new value of B may have to be tried.

Summary of Formulas Proposed by Other Writers.

Maximum Water-Hammer.—All formulas for instantaneous closing agree, however they may be deduced. At first sight they may seem to differ, but by simplifying they all reduce to the same form. In several standard books, however, typographical errors and a confusion between feet and inches lead to erroneous results.

Ordinary Water-Hammer.—Several other formulas have been proposed—deduced from work and energy, or impulse and momentum, or from assumption and experiment.

The following formula is given by D. W. Mead, M. Am. Soc. C. E.,* as a general expression for water-hammer. It is also given by Mr. Uhl in his paper,† but he considers it less accurate than Allievi's formula.

Assuming a head, h, caused by the closing of the gate and acting throughout the time, T, by impulse and momentum

$$P\,Wh\,T = \frac{W\,L\,P\,V}{g}$$

$$h = \frac{L\,V}{g\,T} \qquad (47)$$

* "Water Power Engineering," p. 450.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 34, p. 347.

This is erroneous, as it neglects the time, $\frac{2\,L}{a}$, required for the pressure wave to travel up the pipe before h becomes constant. This error is most important in small values of T, and for such values may yield results 50% too small.

This error is negligible, however, for larger values, and for such the formula is sufficiently accurate. It also neglects water entering and leaving the pipe, and thereby introduces a slight error for very large values of T.

This formula may be obtained at once as a special case of the general formula

$$h = \frac{L\,V}{g\left(T - \frac{L}{a}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

by neglecting $\frac{L}{a}$ when T is comparatively large. It is evidently 50% in error, however, when $T = \frac{2\,L}{a}$.

I. P. Church, Assoc. Am. Soc. C. E., quotes the following formula of Joukovsky,* and Mansfield Merriman,† M. Am. Soc. C. E., also gives it, but states that it is not applicable to all cases.

It is derived as follows: We have shown (Formula (7)) that the maximum water-hammer is $h = \frac{V\,a}{g}$. In this case the gate is closed instantaneously, or before any reflected waves have arrived to cut down the pressure. These arrive in the time, $\frac{2\,L}{a}$, so it has been erroneously assumed that, if the gate is closed in a longer time, the rise of pressure is proportionally less. Writing the proportion

$$h : \frac{V\,a}{g} :: \frac{2\,L}{a} : T,$$

$$h = \frac{V\,a}{g}\,\frac{2\,L}{a\,T} = \frac{2\,L\,V}{g\,T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

This formula is wrong, theoretically, and does not agree with practice or experiment, except for the special case where $T = \frac{2\,L}{a}$, when it becomes the common formula, $h = \frac{V\,a}{g}$. It can be used for small values only, and for larger values may be almost 100% in error.

* "Hydraulic Motors," p. 209.
† "Hydraulics," p. 391.

Surge in Surge Tanks.—Mr. R. D. Johnson has derived the following equation for maximum surge in surge tanks:

$$S = \sqrt{\frac{P\,L\,V^2}{A\,g} + F^2}$$

$$t_1 = \frac{\pi}{2}\sqrt{\frac{L\,A}{g\,P} + \frac{C^2\,V^2\,A^2}{P^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

where $F = C\,V^2$.

The writer has derived, for an approximation:

$$S = \sqrt{\frac{P\,L\,V^2}{A\,g} + \frac{F^2}{9} + \frac{1}{3}F} \quad \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

Mr. D. W. Mead has the following formulas:

Neglecting friction, but including governor action, S, for a drop in level is given by

$$S^2 - 2\,Y\,S = -\frac{2\,P}{A}\left(\frac{L\,V^2}{2\,g} + \frac{Y\,T\,V}{\pi}\right) \quad \ldots\ldots\ldots\ldots (51)$$

where $$T = \pi\sqrt{\frac{A\,L}{P\,g}}.$$

To this F must be added to approximate friction. . . . . (52)

Including friction, Mr. Mead gives for upward surge:

$$S = \sqrt{\frac{P\,L\,V^2}{A\,g} - \frac{F\,T\,Q}{3\,A}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (53)$$

His most exact formula is

$$S^2 - 2\,(Y - F)\,S = -\frac{2\,P}{A}\left(\frac{L\,V^2}{2\,g} - C\,V^3\,(0.37\,T_1) + \frac{Y\,T_1\,V}{\pi}\right) \ldots (54)$$

Where $$C = \frac{F}{V^2}$$

$$\frac{\tau}{2} = T = \pi\sqrt{\frac{L\,A}{P\,g}} \text{ (no friction or governor)}$$

$$G = \sqrt{\frac{P\,L\,V^2}{A\,g}} \text{ (as above)}$$

J = answer to Mead's Equation (51) (no friction)

$$T_1 = \frac{J}{G}\,T.$$

Economical Size of Penstocks.—Using the same methods, a similar formula for economical penstock size can be developed, assuming that the pipe thickness varies as the diameter squared. This is sometimes theoretically true, but practically long lengths of the pipe are

made of constant thickness, and the allowance of metal for rust and water-hammer is independent of the head. Therefore, the foregoing formula is preferable, as it is more simple.

Numerical Examples.

Maximum Water-Hammer.

Formula (*A*).—A penstock, 10 ft. in diameter, ½ in. thick, and 400 ft. long, has water flowing in it at 6 ft. per sec. The gate is under a 100-ft. head. The maximum rise of pressure that can develop due to closing the gate is (Formula (*A*)):

$$D = 10$$
$$b = 0.041$$
$$L = 400$$
$$Y = 100$$
$$V = 6$$

$$\frac{D}{b} = \frac{10 \times 12}{\frac{1}{2}} = 240$$

$$h = \frac{145\ V}{\sqrt{1 + 0.01\ \frac{D}{b}}} = \frac{145 \times 6}{\sqrt{1 + 0.01 \times 240}}$$

$$= \frac{870}{\sqrt{3.4}} = 465 \text{ ft.}$$

To this must be added the static head (100 ft.) making a total pressure at the gate of 565 ft., or 245 lb. per sq. in.

Ordinary Water-Hammer.

In order to test out and show the limitations of the four formulas for ordinary water-hammer, three cases are worked out:

Case I.—The foregoing data are used, with a closing time of 5 sec.

Case II.—The foregoing data are used, with a closing time of 0.32 sec.

Case III.—The foregoing data are used, with a closing time of 0.32 sec., but the head, *Y*, is taken as 400 ft., and not 100 ft.

Case I.

Using the foregoing data, with a closing time of 5 sec.

Formula (*B*).—By Formula (*B*),

$$h = \frac{L\,V}{g\left(T - \frac{L}{a}\right)},$$

where, for steel pipe, $a = \dfrac{4\,660}{\sqrt{1 + 0.01\frac{D}{b}}}$,

$$a = \frac{4\,660}{\sqrt{3.4}} = 2\,500, \frac{L}{a} = \frac{400}{2\,500} = 0.16,$$

$$h = \frac{400 \times 6}{32.2\ (5 - 0.16)} = 15.5 \text{ ft.}$$

Therefore, the total maximum head is 115.5 ft.

Formula (*C*).—From Formula (*C*),

$$h = \frac{N\,Y}{2} + Y\sqrt{\frac{N^2}{4} + N}$$

$$N = \left(\frac{L\,V}{g\,T\,Y}\right)^2 = \left(\frac{400 \times 6}{32.2 \times 5 \times 100}\right)^2 = 0.0222,$$

$$h = \frac{0.0222 \times 100}{2} + 100\sqrt{\frac{0.0222^2}{4} + 0.0222} = 16.1 \text{ ft.}$$

Formula (*47*).—Formula (47) gives

$$h = \frac{L\,V}{g\,T} = \frac{400 \times 6}{32.2 \times 5} = 15 \text{ ft.}$$

Formula (*48*).—Formula (48) gives twice this, or

$$h = \frac{2\,L\,V}{g\,T} = 30 \text{ ft.}$$

Case II.

Taking the same data, with a closing time of exactly $\dfrac{2\,L}{a} = 0.32$ sec., we know we should get the same results as from Formula (*A*), maximum water-hammer (465 ft.). Using the different formulas, we get the following:

Formula (*B*) $h = \dfrac{400 \times 6}{32.2\ (0.32 - 0.16)} = 465$

" (*C*) $N = 5.4, h = \dfrac{540}{2} + 100\sqrt{\dfrac{5.4^2}{4}} + 5.4 = 630$

" (47) $h = \dfrac{2\,400}{32.2 \times 0.32} = 232$

" (48) $h = \dfrac{4\,800}{32.2 \times 0.32} = 465$

Case III.

Taking Case II, with a head of 400 ft., the only formula which will be altered is Formula (*C*).

With $Y = 400$, $N = 0.34$, $h = \frac{0.34 \times 400}{2} + 400\sqrt{\frac{0.34^2}{4} + 0.34} = 312.$

Table 1 is a comparison of the foregoing.

TABLE 1.—COMPARISON OF RESULTS.

Case.	Value of *T*.	Value of *Y*.	Formula.	Head. in feet.	Percentage of difference.
I	5 sec.....	100 ft.	(*B*) Correct formula.....	15.5	0
			(*C*) Allievi.....	16	+ 3%
			(47).....	15	− 3%
			(48).....	30	+ 94%
II	0.32 sec $= \frac{2L}{a}$...	100 ft.	(*A*) or (*B*) Correct formula.....	465	0
			(*C*) Allievi.....	630	+ 35%
			(47).....	232	− 50%
			(48).....	465	0
III	0.32 sec.....	400 ft	(*A*) or (*B*) Correct formula.....	465	0
			(*C*) Allievi.....	312	− 34%
			(47).....	232	− 50%
			(48).....	465	0

The comparison of numerical results in Table 1 for three different cases brings out the limitations of the various formulas very clearly, and establishes the following general rules:

Formula (*B*) is the best for general use.

Allievi (Formula (*C*)) is only good for comparatively slow closing times. For quick closing his formula is incorrect, and may give results too high or too low.

Formula (47) is useless for general application, but is a good check for slow closing of gates.

Formula (48) is useless for general application, but is a good check for quick closing of gates.

Surge in Surge Tanks.

Taking the following case: A pipe, 5 ft. in diameter, and 13 000 ft. long, is passing 112 cu. ft. of water per sec. at full load. There is a total loss of 20 ft. in the pipe running full. A surge tank 14 ft. in diameter is to be installed. The draw-down of the reservoir is 30 ft.

The average head on the wheels is 440 ft. Determine the minimum height of the surge tank required.

$Q = 112$;

$P =$ pipe area $= 19.6$ sq. ft.;

$L = 13\,000$ ft.;

$A =$ surge tank area $= 154$ sq. ft.;

$F = 20$ ft.;

$V =$ full load velocity $= \frac{112}{19.6} = 5.7$ ft. per sec.;

$Y = 440$ ft.

Insert these numerical values in the various formulas, and, solving for S, we get the results shown in Tables 2 and 3.

TABLE 2.—SURGE IN SURGE TANKS. RESULTS BY SEVERAL FORMULAS.

Conditions.	Formula.	Maximum surge in feet, S.	Percentage of difference.
Neglecting friction	(*D*) Common formula	41	0
	(51) D. W. Mead (First approximation including governor action)	45	+7%
Including friction	(*G*) D. L. Webster	46	0
	(49) R. D. Johnson	46	0
	(50) Writer's approximate formula	48	+6%
	(*F*) D. L. Webster (First approximate)	48	+6%
	(54) D. W. Mead (Most exact, friction and governor action)	50	10%
	(*E*) D. L. Webster (Second approximate)	61	35%
	(52) D. W. Mead (First approximate)	65	43%
Rise above reservoir level	(*G*) D. L. Webster	26	0
	(53) D. W. Mead	29	+12%

TABLE 3.—SURGE IN SURGE TANKS.

TIME FOR COMPLETE CYCLE; AND TIME TO MAXIMUM DRAW-DOWN (STARTING UP).

	Formula.	Time.	Percentage of difference from Formula (36).
Time for complete cycle	(36*A*) D. L. Webster	$\tau = 365$	0
	(54) D. W. Mead (including friction)	$\tau = 390$	+7%
	(51) D. W. Mead (no friction)	$\tau = 354$	−3%
Time to maximum draw-down (starting up)	(36) D. L. Webster	$t_1 = 102$ sec.	.....
	(49) R. D. Johnson	$t_1 = 99$ sec.	.....

From the results in Tables 2 and 3, using Webster's or Johnson's formula, the tank must rise 26 ft. above maximum reservoir level, and must extend 46 ft. below minimum reservoir level. Therefore, if the surge tank is 14 ft. in diameter, it must be $46 + 30 + 26 = 102$ ft. high.

Economical Penstock Size.

A development is to be built on a small stream by creating a very large reservoir which will completely equalize the flow of the stream; the water is to be conducted to the surge tank and penstocks by a single wood stave pipe, 13 000 ft. long. The head is 440 ft.; the average flow through the pipe while the plant is in operation is 112 cu. ft. per sec., which develops about 3 000 kw. There are two 2 000-kw. units. The plant is to be used as a base load plant, and the load is to be very nearly constant, therefore q may be assumed as 112 cu. ft. per sec.

The plant develops about 26 000 000 kw-hr. of primary power yearly, which, sold at 1 cent, will give a gross income of \$260 000, or \$590 per ft. of fall.

A velocity of 6 ft. per sec. is first assumed, which would require a 5-ft. pipe. Such a wood pipe, in place and back-filled, is estimated at \$10 per ft., or \$2 per ft. of diameter.

Inserting these values in the formula:

$q = 112$

$i = 590$

$C = 113$ (assumed constant for Chezy formula)

$p = 15\%$ (assumed for profit, maintenance, depreciation, etc.)

$B = \$2$.

$$d = \sqrt[6]{\frac{3\,250\,q^2\,i}{C^2\,p\,B}} = \sqrt[6]{\frac{3\,250 \times 112^2 \times 590}{113^2 \times 15 \times 2}}$$

$$= \sqrt[6]{6\,250} = 4.29 \text{ ft.}$$

A 4 ft. 3½-in. pipe is most economical, or, to make a more common size and err on the side of better regulation, say, a 4½-ft. pipe.

The writer wishes to acknowledge gratefully the help of Dr. D. L. Webster and the criticisms of George F. Swain, Past-President, Am. Soc. C. E., and Mr. J. F. Vaughan, in preparing this paper. The following books were also consulted.

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THE DIFFERENTIAL SURGE TANK.

Discussion.*

By Roy Taylor, Esq.

Mr. Taylor.

Roy Taylor, Esq. (by letter).—In comparison with the great advancement made during recent years in all branches of hydro-electric work, progress in the matter of regulating long pipe lines seems to have remained practically stationary. With the exception of improvements in some details, regulating works to-day are built the same as those of ten years ago. In many cases even the idea of regulating the long water columns has been abandoned, and the fluctuations of draft have been taken care of by nozzle deflectors and waste weirs. The only explanation of this lack of progress seems to be that until recently only the attractive developments were undertaken where long pipe lines were not required. Troubles with electrical apparatus in the past were so common, and interruption in service so frequent that occasional poor hydraulic regulation was of small importance in comparison. To-day conditions are rapidly changing, electrical devices are far more reliable, interruptions are becoming more infrequent, and the desirability of satisfactory regulation without waste of water will soon become a necessity.

The most common type of hydraulic regulator now in operation, and the one most productive of good results, is an open storage tank or reservoir at the lower end of the pipe line. In the future, as the market for power grows, more and more of the less desirable developments requiring long pipe lines will attract attention, and this form of regulation will have even greater claim to importance. A certain few, however, where the ground at the lower end of the pipe line is

*This discussion (of the paper by Raymond D. Johnson, Esq., published in this number of *Proceedings*, and to be presented at the meeting of December 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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too low to make an open tank practicable, will have to depend on the less satisfactory though perfectly feasible device of air tanks. The differential principle is adaptable advantageously to them as well and is treated mathematically in Mr. Johnson's paper of 1908.

The form of open storage now in general use is a simple tank or reservoir with an unrestricted flow between it and the conduit. Many of these have been provided with overflows which although they waste water, are necessary to carry away excess water following shut-downs. In some cases, as for example that of the surge chamber at San Francisquito, Power Station No. 1, on the Los Angeles Aqueduct, it has been deemed necessary to go to great expense to provide sufficient capacity to insure good regulation. Many others have been intended to regulate under certain load changes, yet when put in operation have proved unsatisfactory, due probably to the coincidence of change of load with the periodic surges in the tank. Some find it possible to go on operating under these unstable conditions with poor regulation, and a few have been forced through expensive accidents to remodel their whole regulating works.

That practically all these disadvantages of the simple tank can be overcome, and the difficulties now arising in operating with some of them can, in a large measure, be relieved by the use of the differential principle, there is not the slightest doubt.

For equal regulating ability under the same load changes, the differential surge tank can be made (1) much smaller in diameter than the simple tank; (2) it does not have to be carried to such height as the latter, often saving an expensive spillway and waste of water; and (3), most important of all, it is proof against synchronous load changes, which, however small, are likely to carry the water in the ordinary simple tank beyond bounds, or at least to cause serious surging. The probability that load will be thrown on and off many times synchronously with the natural period of the water wave in the tank, may be slight, yet, as a possibility, it is always present, especially as the loads may be of almost any size and do not have to come directly in step with the water wave to cause serious surging. An example of the comparative effect of the same synchronous load changes on the operation of a differential and simple tank of equivalent capacities has been computed by the method of arithmetic integration, and is shown graphically in Fig. 3. It may be interesting to know that this diagram is taken from an actual study made for a proposed development. The surge in the simple tank soon increases beyond bounds, but, in the differential tank, it increases slightly until the choking action is enough to prevent further increase in the surging; and, if the same load changes are continued periodically, the surging remains at constant amplitude.

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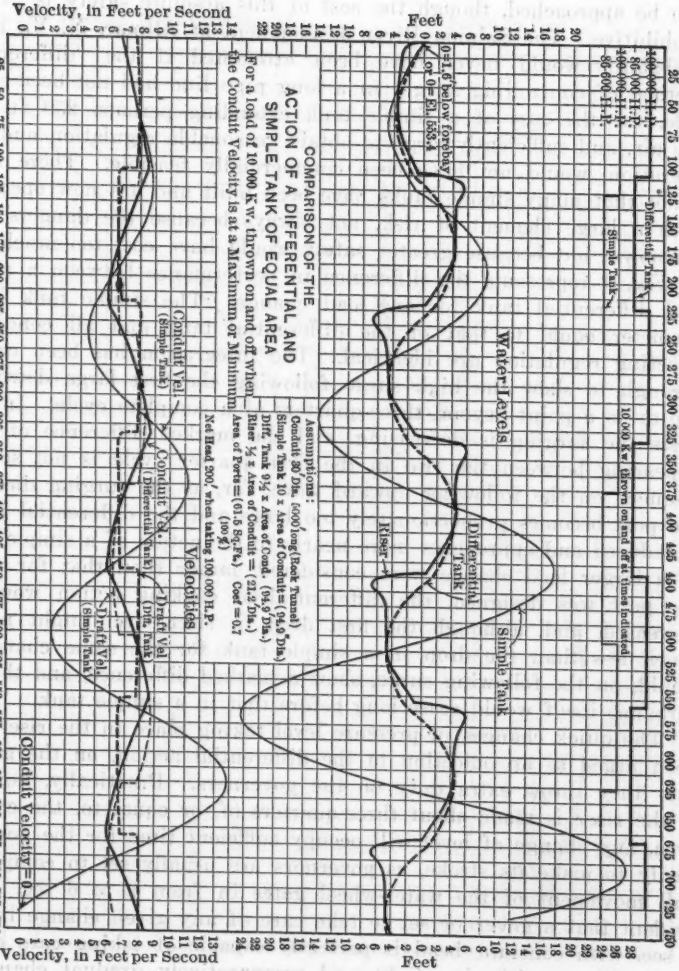

FIG. 3.

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Although Fig. 3 represents very well the usual difference in performance to be found between the two types of tank, it must not be concluded that every simple tank will behave in this manner, for, by increasing the diameter sufficiently, a surging of constant amplitude can be approached, though the cost of this absolute safety is usually prohibitive. The Salmon River development at Altmar, N. Y., in all probability would never have been attempted if the "differential" method of controlling surges in a long pipe line had not been available, for the cost of a simple tank extending perhaps 250 ft. into the air, and sufficiently large to obtain reasonable regulation and safe operation, would not have been commercially feasible. There is no doubt that many simple tanks have been built and are now operating with a large element of risk, not always because the dangers were unknown, but because greater safety would have cost too much.

Fig. 4 represents the difference in performance between a simple and differential tank for one load change. The simple tank has a diameter equal to that of the differential tank and all conditions effecting regulation are identical. The integration has been carried through to show the high surge following the first large downward dip or to a point beyond three-quarters of a complete cycle. In both cases the surging is decreasing, and the marked difference of 5 ft. appearing between the two in the first quarter cycle is not so pronounced on the following upward wave, for, as the surging dies out, the port becomes proportionately too large and the differential action is reduced and approaches more nearly to the action of a simple tank. If a larger load change were considered, larger than that for which the port was designed, the differential or choking action would be increased, and, although the first drop in riser level might not be much less than the drop in a simple tank for the same change, it would, on the following surge, show a marked difference, and the disturbance itself would cease long before that in a simple tank.

The quick changes in pressure level taking place in the riser have been urged as an objection to the differential action, on the ground that they throw extra duty on the governors. Practically, however, if the riser is made about three-quarters of, or equal to, the conduit area, any change of level will occupy sufficient time for the governor easily to make its stroke, as governors are usually set to complete a full movement of the water-wheel gates in from 2 to 6 sec. It is evident that a governor set to take care of any given change in, say, 4 sec. with constant head is perfectly capable of taking care of the same change with the slight and comparatively gradual change in head which occurs in the riser in this short time.

Passing then to a detailed consideration of the differential principle, it is the primary object of this discussion to ascertain the degree of approximation attained in the use of the formulas derived by

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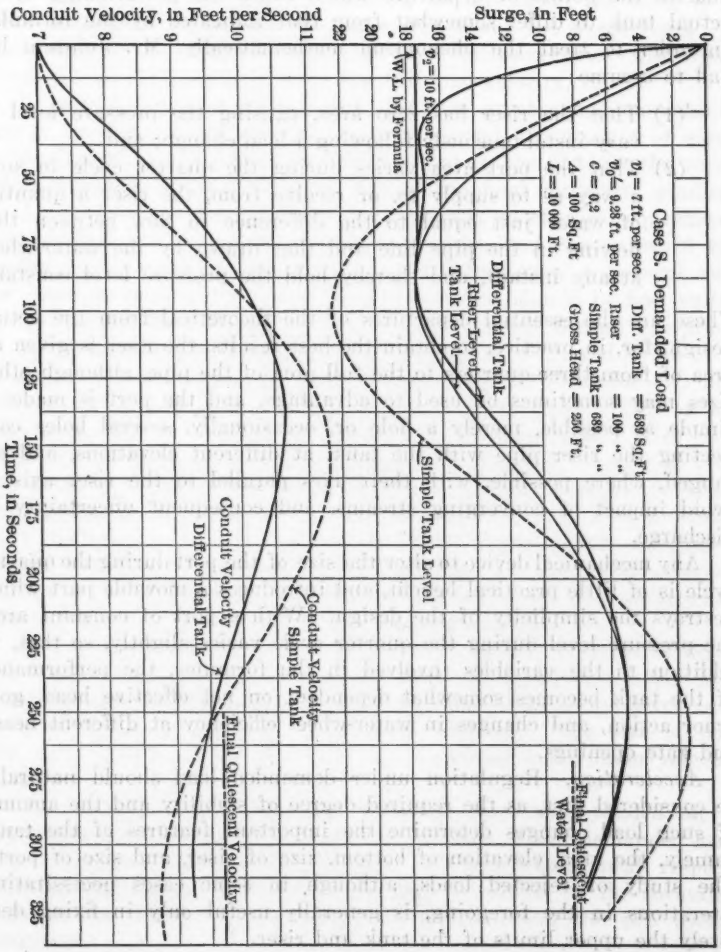

FIG. 4.

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Mr. Johnson, and to demonstrate their value in arriving, in a comparatively short time, at the general design of a tank for any desired degree of regulation.

Before considering specific numerical cases, it may be well to emphasize the points of departure which cause the performance of an actual tank to differ somewhat from that indicated by the formulas. In order to treat the phenomena mathematically, Mr. Johnson has had to assume:

(1) That the riser has zero area, causing the pressure level to vary instantaneously following a load change; and

(2) That the port area varies during the quarter cycle in such a way as to supply to, or receive from, the riser a quantity of water just equal to the difference in flow between that flowing in the pipe line and that drawn by the water-wheel at any instant, and thereby hold the pressure level constant.

These are the essential departures of the theoretical from the actual design, for, in practice, to obtain the best results, the riser is given an area of from three-quarters to the full area of the pipe, although other sizes may sometimes be used to advantage, and the port is made as simple as possible, merely a hole or, occasionally, several holes connecting the riser pipe with the tank, at different elevations, and arranged, where possible, with their axes parallel to the riser axis to avoid impact of converging streams, and consequent uncertainty of discharge.

Any mechanical device to alter the size of the port during the quarter cycle is of little practical benefit, and introduces a movable part which destroys the simplicity of the design. With a port of constant area, the pressure level during the quarter cycle varies slightly, so that, in addition to the variables involved in the formulas, the performance of the tank becomes somewhat dependent on net effective head, governor action, and changes in water-wheel efficiency at different heads and gate openings.

Acceleration.—Regulation under demanded load should naturally be considered first, as the required degree of stability and the amount of such load changes determine the important features of the tank, namely, the area, elevation of bottom, size of riser, and size of ports. The study of rejected loads, although in some cases necessitating alterations in the foregoing, is generally useful only in fixing definitely the upper limits of the tank and riser.

Perhaps the best way to illustrate the use of Mr. Johnson's formulas is to consider a general method of procedure in arriving at a design. The length of pipe line, L, is the one factor most often determined at the start. The area, A, of the pipe line may be fixed, within comparatively narrow limits, by questions of economy. Assum-

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ing that it is thus fixed, the value of c becomes a matter of judgment, depending on a knowledge of the frictional coefficient of the pipe material and the probable losses of head due to bends, etc. In fixing the elevation of the tank bottom, this should be given the highest value that could reasonably be expected, for the higher the value of c, other conditions remaining the same, the lower the surge level. A knowledge of the water-wheels and their characteristics will make it possible to estimate accurately the full load velocity, v_2.

It remains then to decide, first, for how great a sudden velocity change good regulation is desirable; and second, the amount of surge to be allowed, or the degree of stability, that is, the distance the water level should be allowed to drop beyond the final quiescent gradient level. The wide variation in load requirements, mechanical equipment, adaptability of the site for regulating works, and the value of the coefficient, c, make it impossible to formulate even a general rule for deciding these points. The design, however, should be best suited to the largest load change likely to occur during normal operation.

Adopting, then, a certain velocity change, $v_2 = v$, and value of k, y_1 and z can be computed. Substituting them in Equation (4) and solving for F will give the correct area for the tank. This is net tank area, and is exclusive of the area of the riser pipe. With very little further work, y_1 may be varied and its effect on the size of tank determined. Experiments with different velocity changes or other assumptions as to the probable value of c, can be tried, even a change in A, the area of the pipe line, may be found to give a more economical design. In short, by the use of the formulas, the whole field of possibilities can be investigated without needless work, and the combination adopted which is best suited to the exigencies of the case.

As for the elevation of the tank bottom, a liberal allowance of depth below the calculated surge should be made for occasional larger load demands and for absolute insurance against lower surge levels due to a coefficient of friction higher than expected. The size of port and upper limits of the tank will be considered later under rejected load.

To illustrate how closely the formulas may be depended on to approximate the actual performance of a design, under demanded load, four numerical cases have been selected, differing widely in all respects affecting regulation. They have been first calculated by the formulas, then by the method of arithmetic integration, and the two results compared.

The assumptions are given in Table 1.

In using the formulas, it is necessary to deal entirely with velocity changes, though, fundamentally, the load changes, in terms of power, are what influence the design. To change one into the other is comparatively simple, if it is remembered that v_2 is the conduit or draft

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TABLE 1.

Case.	Length of pipe line, in feet. L.	Area of pipe, in square feet. A.	Gross head, in feet. H.	Frictional coefficient. c.	Velocity, in feet per second. v_1.	Velocity, in feet per second. v_2.	Stability, or k.
S	10 000	100	250	0.2	7	10	1.7
T	6 440	200	300	0.05	6	20	2.2
U	32 200	100	1 000	0.56	7.2	9	1.15
V	3 220	113	300	0.0555	12	24	1.05

velocity at the bottom of the surge, or end of the quarter cycle. The head at that time being less, the velocity change, $v_2 - v_1$, represents more than the actual load change by an amount depending on the size of surge and the net effective head. Expressing the relation mathematically:

$(H - c\,v_1^2)v_1$ = original power before load change,

$(H - c\,v_1^2)v_0^1$ = demanded power at instant load is thrown on and before riser level has dropped,

v_0^1 being the initial draft velocity for the new load, in terms of the conduit velocity.

$(H - c\,v_1^2 - y_1)v_2$ = demanded power at the end of the quarter cycle with riser level at the bottom of the surge.

As the demanded power must be held constant after the initial change,

$$(H - c\,v_1^2)v_0^1 = (H - c\,v_1^2 - y_1)v_2$$

Or the actual load change in horse-power can be expressed as follows:

$$\frac{A\,v_0^1\,(H - c\,v_1^2) \times 62.5 \times \text{Efficiency}}{550} - \frac{A\,v_1\,(H - c\,v_1^2) \times 62.5 \times \text{Efficiency}}{550}$$

and, if the water-wheel efficiency is assumed to be constant, as has been done throughout this discussion, the load change, in horse-power is

$$\frac{A\,(H - c\,v_1^2) \times \text{Efficiency}}{8.8}\,(v_0^1 - v_1).$$

The results obtained for the trial cases by Mr. Johnson's formulas, and using his nomenclature, are given in Table 2.

The same cases were then computed by using arithmetic integration, following the changes in velocity and water levels second by second. A head had to be assumed and the velocity change at the start corresponding to the load change calculated. As the head at the quarter

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cycle was unknown without a trial, that given by the formulas in Table 2 was used, and the value of v_0^1 computed from $v_2 \times$ net head quarter cycle $= v_0^1 \times$ net head at start. The initial velocity change is then $v_0^1 - v_1$, and as the pressure level in the riser drops, v^1, the draft velocity, increases to a value, v_2, to hold the power constant. The resulting surge was found to be near enough to the assumed value to require no revision of the value v_0^1.

TABLE 2.

Case.	Z.	F, in square feet.	y_1, in feet.	t, in inches.	a_0, in square feet.	a_1, in square feet.
S	11.65	589	17.35	79	8.97	6.49
T	28.87	2 800	40.00	89	55.2	43.5
U	9.24	709	18.82	217	5.18	2.32
V	24.43	2 088	25.2	134	33.7	9.25

The friction and velocity head in the riser and the inertia of the water in both tank and riser were neglected, and 3 sec. were allowed for the governors to open up the turbine gates. A riser area equal to A was used, and the water-wheel efficiencies were assumed to remain constant. The work was first carried through using a port equal to a_0 and then, by trial and error, the best size of port was determined and the resulting performance calculated. The best size is one which gives an initial drop in the riser level equal to the final level of tank and riser at the end of the quarter cycle. It must be understood that all port areas in Mr. Johnson's formulas, as well as any figures for port area appearing in this discussion, assume a 100% coefficient of discharge, so that the actual ports when built must be increased to allow for losses.

The results are given in Table 3, together with those obtained by the formula, for the sake of comparison.

TABLE 3.

Case.	Size of port, a_0, in square feet.	Surge, y_1, with port $= a_0$, in feet.	Best size of port, a, in square feet.	Surge, y_1, with port $= a$, in feet.	Surge, y_1, by formula, in feet.	Percentage of increase of surge over result by formula.
S	8.97	18.20	8.1	17.51	17.35	0.9
T	55.2	41.49	52.6	40.52	40.00	1.3
U	5.18	20.16	4.36	19.14	18.82	1.7
V	33.7	27.29	30.0	26.45	25.2	5.0

The results obtained by formula and those computed by arithmetic integration with a port area equal to a_0, as well as the best size of port, were plotted together for comparison, and are shown in Figs. 5, 6, 7,

Mr. Taylor.

and 8. Only the riser levels are shown, as they alone affect the regulation directly. In all cases the tank and riser levels coincide at the beginning and end of the quarter cycle.

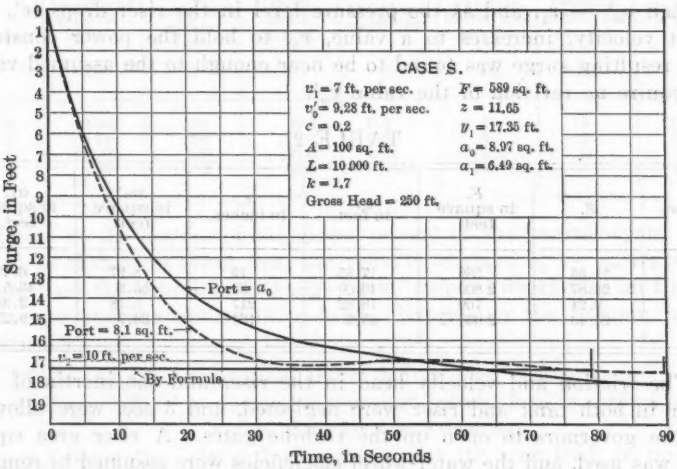

FIG. 5.

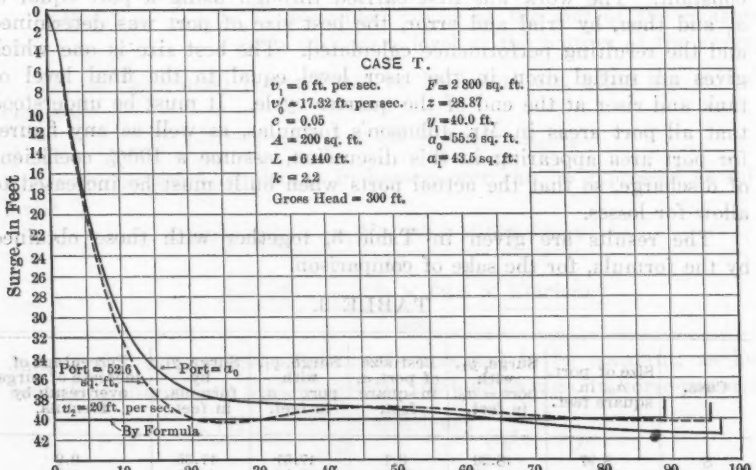

FIG. 6.

How closely the formulas approximate the performance with the correct size of port can readily be seen from the diagrams. In Cases S, T, and U, they almost exactly agree, and in only one instance, Case V, do they differ by as much as 2 per cent. In computing this

particular case, if the results of the formula are revised to be identical with the conditions of the arithmetic integration, making $v_2 = 24.10$ instead of 24.00, y_1 will be found to be 25.45 ft. instead of 25.2 ft., so that the computed result really only exceeds that obtained from the formula by less than 4% instead of 5% as given in Table 3. Looking at it in another way, if the surge, y_1, in the formula is made equal to that

Mr. Taylor.

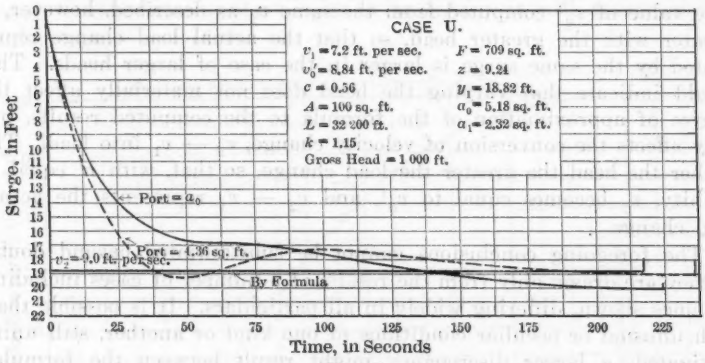

FIG. 7.

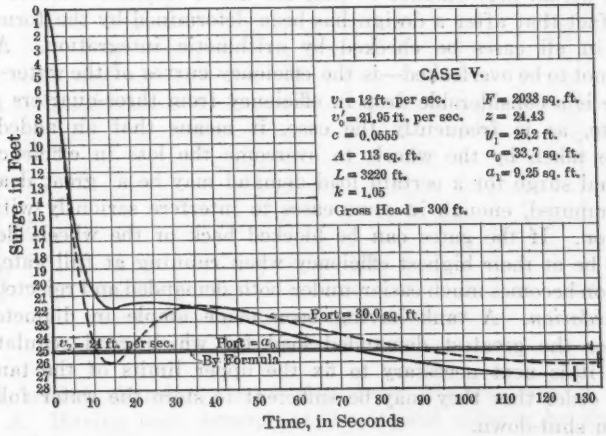

FIG. 8.

obtained by the arithmetic work (26.45 ft.), other values remaining unchanged, the actual load change given by the formula will only exceed the computed load change by 1.5 per cent. This difference is inappreciable when it is remembered that, in the first place, the load change and the value of c for which a tank is designed can only be estimated. This indicates that, as we approach dead-beat stability conditions, or values of k near unity with large load changes, where the difference

Mr. Taylor.

between the theoretical ports a_0 and a_1 is large, the formula drifts slightly away from the actual results. In the other direction, with values of k greater than 1.2 and velocity changes of 10% or more it has been found that the two methods practically coincide. The effect of assuming a larger head was tried in Case S, 650 ft. instead of 250 ft., and the resulting surge was computed as substantially the same. The value of v_0^1 computed from the same v_2 as described, however, is greater with the greater head, so that the actual load change represented by the same surge is larger in the case of larger heads. This would indicate that varying the head does not materially affect the degree of approximation of the formula to the computed results, but only affects the conversion of velocity change, $v_2 - v_1$ into load. The higher the head the greater the load change, so that, with H equal to infinity, v_2 becomes equal to v_0^1, and $v_2 - v_1$ represents the actual load change.

The foregoing conclusions cannot be substantiated beyond doubt, as they are drawn only from the results of a number of cases including the ones shown, differing widely in all particulars. It is possible that, with unusual or peculiar conditions of one kind or another, still uninvestigated, a larger discrepancy might result between the formulas and the actual performance so that too much emphasis cannot be laid on the fact that after a design has been determined by the formulas it should in all cases be checked by arithmetic integration. Another point—not to be overlooked—is the efficiency curves of the water-wheels. If there is a considerable drop in efficiency from three-quarters gate to full gate, as is frequently the case, it means that an added draft must be taken by the wheels to overcome the loss in efficiency, and the actual surge for a certain load demand may be a great deal more than computed, enough in some cases to interfere seriously with good regulation. If the gates can be blocked back or the wheels designed so as to be at their highest efficiency when running at full gate, surge regulation becomes much easier under both demanded and rejected loads.

Retardation.—A tank having been made ample in diameter and depth for the greatest demanded load for which good regulation is desired, it is next necessary to fix the upper limits of the tank and riser in order that they may be sufficient to store the water following a sudden shut-down.

Although the results desirable to attain under demanded load are largely a question of judgment and experience, those due to loads dropped are quite definite. Complete, sudden shut-downs are likely to occur on any plant, on account of line accidents and short circuits, so that the tank should be designed to handle the water if the full plant load is suddenly rejected. The only questions left to judgment are the maximum velocity occurring in the pipe line and the quantity of water likely to escape through relief valves and by water-wheel leakage.

Mr. Taylor.

There is for every specific power development a critical conduit velocity which requires the maximum height of tank. This is shown by Mr. Johnson in Formulas (36), (37), and (38). In some instances this critical velocity, which gives the maximum height of surge, happens to be less than, or not much greater than, the maximum full load conduit velocity. In such cases the critical velocity should be used in determining the height of tank, for it will assure absolute safety against overflow.

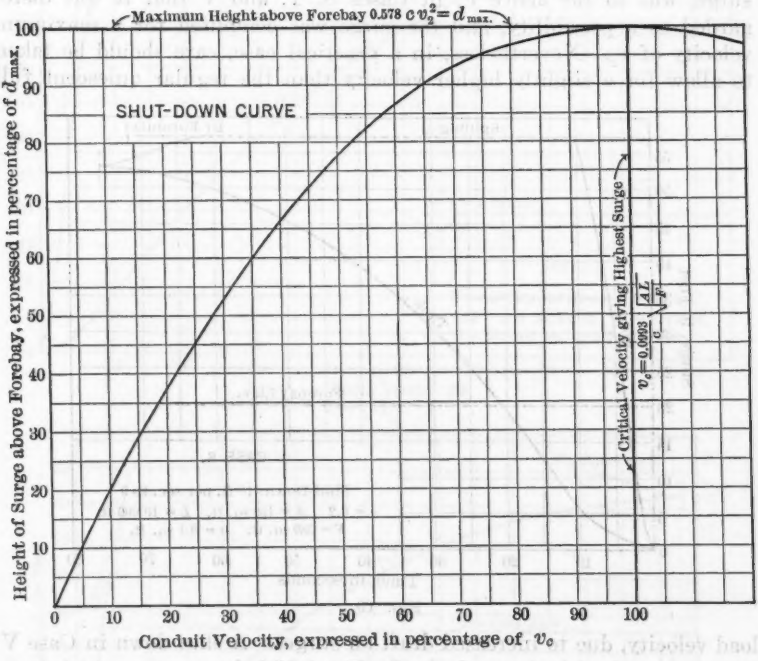

FIG. 9.

A useful shut-down curve has been plotted, and is shown in Fig. 9. Having once determined the critical velocity by the formula, $v = \frac{0.0993}{c}\sqrt{\frac{A\ L}{F}}$, which is merely Formula (36) in a more simple form, the height of surge to be expected at any other conduit velocity can be picked off graphically without additional computation. The height to which the water will rise, in case there occurs a higher conduit velocity than expected, or the effect of varying the assumption as to the value of c, can be determined with quite a saving of time by the use of this curve.

Mr. Taylor.

The great danger to be avoided in computing shut-downs is an error in the assumption of the coefficient, c, which, if smoother than figured, will cause the water at quiescent full load to stand higher and therefore attain a higher surge level than contemplated. For this reason, it is always better, when considering rejected loads, to use the smallest value of c that it is reasonably possible to expect.

The four cases previously referred to under acceleration were also computed for shut-down. The critical velocity giving the highest surge, was so far above v_2 in Cases S, T, and V that it was disregarded as a possibility, and the surge was computed for a maximum velocity of v_2. Nevertheless, in a practical case, care should be taken to allow for a slightly higher velocity than the regular quiescent full

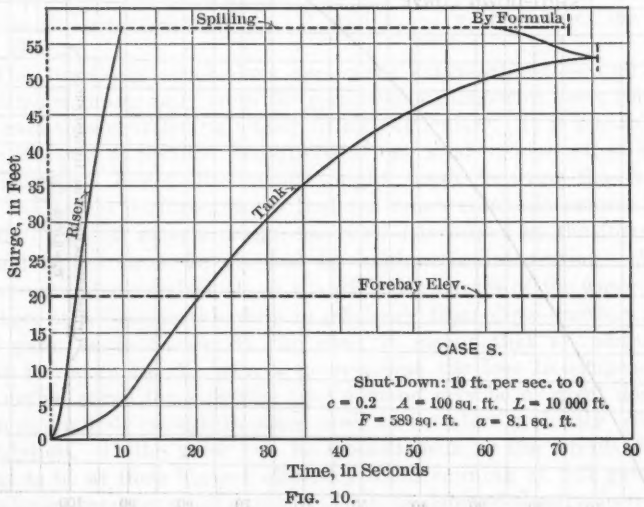

FIG. 10.

load velocity, due to increased draft on surges. A shut-down in Case V was computed with the critical velocity of 23.9 ft. per sec., which happened to be slightly less than v_2. It will be found by trial that, provided c is not made smaller, or the area of tank or conduit changed, no variation in velocity or length of pipe line will result in a higher surge level.

The heights of riser and tank for these four cases were first obtained by Formula (31), solving for the value of y_1 and using F as determined for demanded load. This height, y_1, was adopted for the riser, and the performance was computed by arithmetic integration. The results are shown graphically in Figs. 10, 11, 12, and 13, and the dotted line indicates the time during which the riser was spilling and represents also the height of surge as computed by the formula. The added

back pressure due to the weir height on the top of the riser when spilling, and the inertia and velocity head in tank and riser were neglected. Leakage through the wheels or discharge through relief valves was not considered. Mr. Taylor.

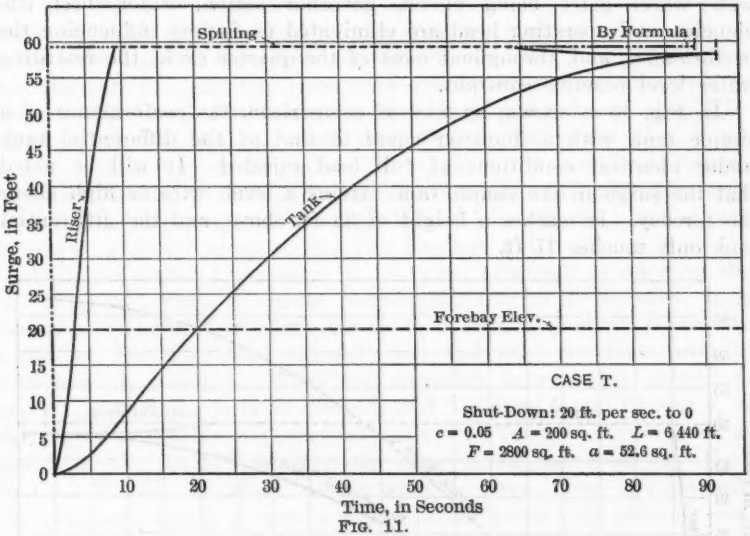

FIG. 11.

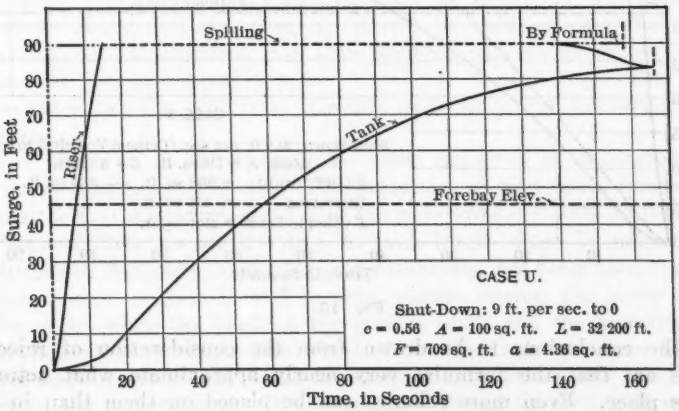

FIG. 12.

In every case the mathematical formulas lean toward the side of safety, giving a somewhat higher tank and riser than necessary. What actually takes place, however, in a shut-down, agrees so nearly with the premises of the theoretical formulas that it is a question in the

Mr. Taylor.

writer's mind whether they may not give results fully as close to what really takes place as those derived by arithmetic integration, unless the latter is computed in very small increments of time, and inertia. Velocity head and weir heights are taken into consideration. The water-wheel gates being closed, governor action, water-wheel efficiencies, and operating head are eliminated as factors influencing the performance, and, throughout most of the quarter cycle, the retarding water level remains constant.

In Fig. 13 is shown, by way of comparison, the performance of a simple tank with a diameter equal to that of the differential tank under identical conditions of full load rejected. It will be noted that the surge in the simple tank attains a level twice as high above the forebay. It reaches a height of 35 ft. above, and the differential tank only touches 17 ft.

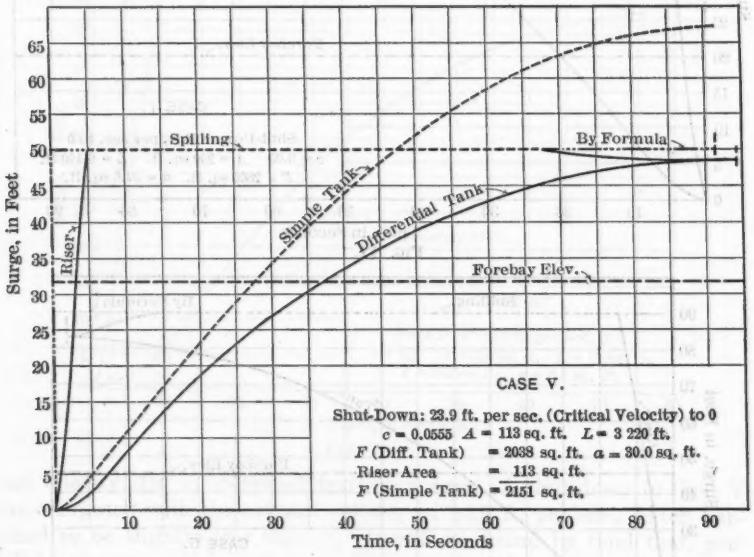

FIG. 13.

The conclusions to be drawn from the consideration of rejected loads are that the formulas very nearly approximate what actually takes place. Even more reliance can be placed on them than in the case of demanded loads. It is the writer's opinion that in the ordinary case they could be relied on to determine the design without further checking; yet, as there is a remote possibility that with certain peculiar conditions a discrepancy might appear, it is advisable to use some form of check or allow a factor of safety in extra height.

Mr. Taylor.

Ports.—The question of ports is very interesting as well as important, for the whole differential principle depends for its success on the choking action of the port. Pressure level changes following a load change are directly affected by the size of the port. Through it must flow the water to keep the riser level from dropping too far on demanded loads or rising too high on ordinary loads rejected.

In order to treat the subject mathematically, Mr. Johnson assumes that the port just supplies or receives enough water from the tank to keep the riser level stationary. To do this it must change size. The variation may be large or small, depending on the values of c, k, v_2, and v_1. It is largest for values of k near unity and load changes equivalent to more than 20% velocity change, and small for large values of k with any load change. It also increases as c and v_2 are increased, and *vice versa*, to zero as c becomes zero.

This fact makes it necessary to use some judgment in selecting the size of port. When a simple unchanging port is to be used, it is desirable to have it of such size as to approximate most nearly the action of the theoretical variable one, and hold the riser level as stationary as possible. When the variation between the values, a_0 and a_1, is small, as it is in Cases S and T (Figs. 5 and 6) size a_0 is quite good enough for all practical purposes. The best size of port, as shown by the dotted line, does not show much improvement. Even in Case V (Fig. 8) where the variation is large, there is little need for refinment unless to save a foot or so of surge. In general, however, when the variation is large, a trial by arithmetic integration should be made, and the area, a_0, should be reduced somewhat for better results. A rule that may save needless trial computation is to assume the port area equal to $a_0 - (0.1 \text{ to } 0.3) \times (a_0 - a_1)$, using the factor 0.3 when the difference is small, varying to 0.1 for large differences.

The reason a_0 so often cannot be much improved is illustrated by the curve shown in Fig. 14. The variation of port area necessary during the first quarter cycle to hold the riser level perfectly stationary has been plotted against the drop in tank level. Case V, under the conditions of demanded load discussed previously, has been used. Most of the change in area, it will be seen, occurs during the latter part of the cycle, when the head on the port is small and the quantity of water necessary to supply the wheels from the tank has been much diminished. During this period a variation from the theory has little effect on the performance. It is important during the first half cycle, when the port is discharging large quantities of water, that its actual size should not be far from the theoretical. The curve shows that during this period the theoretical size does not depart to any great extent from the original area, a_0, and, for this reason, a_0 will often suffice for all practical purposes.

Mr. Taylor.

The performances obtained with the use of various sizes of ports for the same conditions of velocity and load change were computed for Case S, and are shown graphically in Fig. 15. This diagram gives a clear conception of exactly what will take place in any specific case, if the port be made too large or too small. The sizes used range from an area large enough to allow free flow with no differential action to a port of 4.98 sq. ft. area or 0.555 a_0. With too small a port, the surge

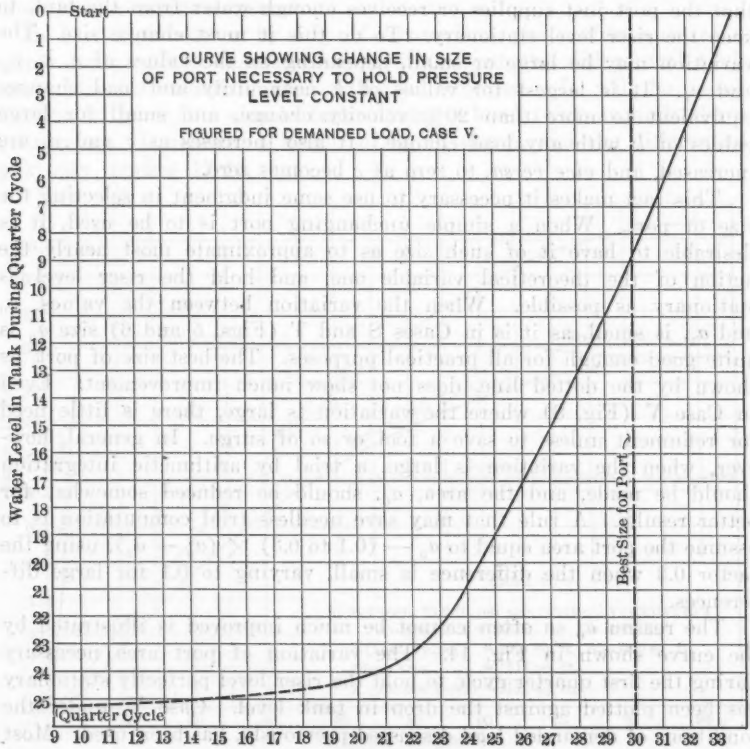

FIG. 14.

is increased on the first dip in riser level, there is more differential action, and at the end of the quarter cycle the water is much nearer the gradient level. Furthermore, if the performance were carried beyond the quarter cycle, the surging would be found to die out more quickly than with larger ports. With ports too large, the first dip disappears, and the water level at the end of the quarter cycle is drawn down, causing not only an excessive surge, but a long continuance of the disturbance following a single load change. The minimum surge

Mr. Taylor.

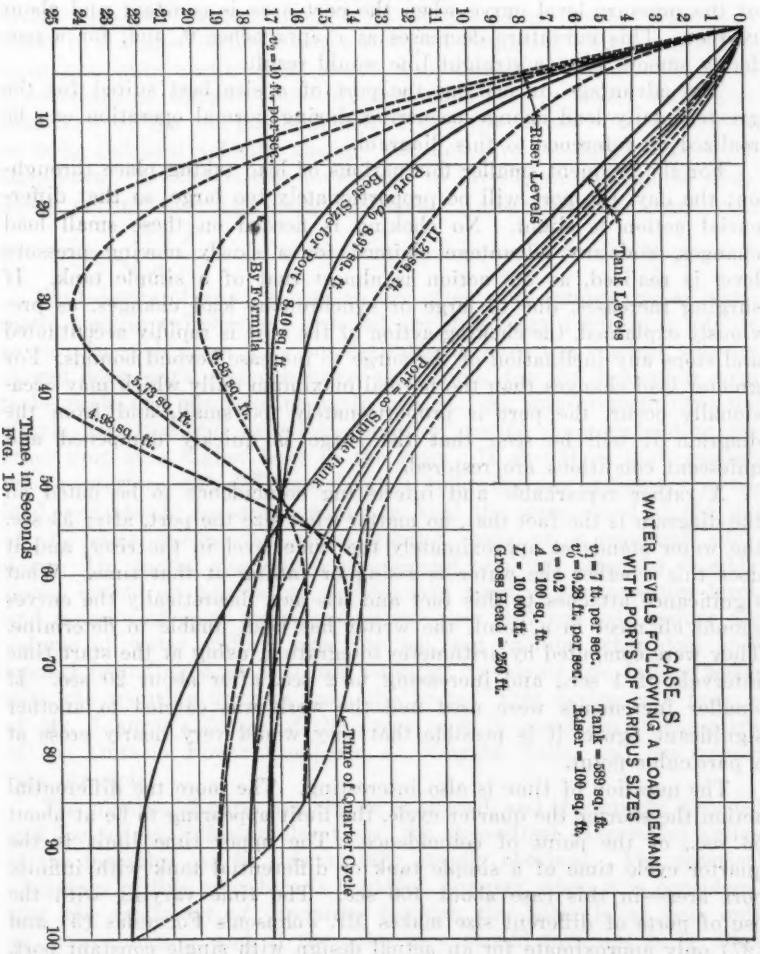

Fig. 15.

Mr. Taylor.

occurs when the first dip in the riser level is equal to the final drop in tank and riser. The port area of 8.1 sq. ft. very nearly fulfills this condition, and is, therefore, the best size of port for the load change under consideration. The slight piling up of the water in the riser between the lowest points of surge always occurs, and is characteristic of the pressure level curve when the port area is constant and about correct. This curvature decreases as c approaches 0, and, for a perfectly smooth pipe, a straight line would result.

The advantage of making the port of a size best suited for the greatest daily load change occurring during normal operation can be realized by reference to this diagram.

For the frequent smaller fluctuations of load taking place throughout the day, the port will be proportionately too large, so that differential action is slight. No choking is needed on these small load changes, and any advantage claimed for a slowly moving pressure level is realized, as the action is almost that of a simple tank. If surging increases, due to large or synchronous load changes, as previously explained, the choking action of the port is rapidly accentuated and stops any inclination of the surge to increase beyond bounds. For greater load changes than the normal maximum daily which may occasionally occur, the port is proportionately too small, and from the diagram it will be seen that the surge is quickly dampened and quiescent conditions are restored.

A rather remarkable and interesting coincidence to be noted on the diagram is the fact that, no matter what size the port, after 53 sec. the water stands at approximately the same level in the riser, and it does this whether the water is rising or falling at that time. What significance attaches to this fact and whether theoretically the curves should all meet in a point, the writer has been unable to determine. They were computed by arithmetic integration, using at the start time intervals of 1 sec., and increasing to 2 sec. after about 20 sec. If smaller increments were used and the work was carried to another significant figure, it is possible that they would very nearly cross at a particular point.

The question of time is also interesting. The more the differential action the shorter the quarter cycle, the limit appearing to be at about 53 sec., or the point of coincidence. The upper time limit is the quarter cycle time of a simple tank or differential tank with infinite port area—in this case about 100 sec. The time varying with the use of ports of different size makes Mr. Johnson's Formulas (3) and (27) only approximate for an actual design with single constant port. The extent of the departure from the actual time may best be judged from Figs. 5 to 8, which seem to indicate a variation of not more than 10% in extreme cases. As a knowledge of the exact time is unimportant, the formulas are close enough for all practical use.

Mr. Taylor.

Thus far, only port areas determined for demanded loads have been considered, as in nearly every case this requirement for good regulation fixes the size of port. One error, however, must be guarded against, that of making the port area too large for shut-down conditions and thus causing the riser level to draw down during the quarter cycle, with the accompanying loss of retarding head. This will result in a higher ultimate surge than contemplated. Such a predicament can only occur when severe and unusual requirements make it necessary to compute suddenly demanded velocity changes of more than 50% of the full-load velocity. For large values of k, even a 70 or 80% full-load velocity change will not give too large a port for shut-down. A safe course, however, is to investigate a complete shut-down performance by using arithmetic integration in every case where $c\, v_2 \sqrt{\frac{2\, g\, F}{A\, L}} + \frac{1}{10}$ is greater than $2\, r$ (see Formula (34)).

Shut-downs of any description are often only temporary, as automatic electric switches trip out and are closed again within a minute or so. In such cases it is not desirable to have the conduit velocity slowed down with as much haste as possible, for the demand is soon to be made and the velocity again needed. An improvement over the simple port that will help this condition can be made by adding another port at a higher level in the riser. It should be high enough not to interfere with differential action following any ordinary rejected load, and should be small enough to hold the water level at the top of the riser during the entire quarter cycle of a complete shut-down. On partial shut-downs this second port will often be of additional benefit in preventing unnecessary rises of the pressure level, and a series of small ports around the riser will not only reduce the volume of water spilling over the top, but cushion what does spill against pounding down into the tank, by traversing the falling water with the spouting streams from the ports. This device is a feature of Mr. Johnson's design of the surge tank in operation at the plant of the Ontario Power Company, at Niagara Falls, and has proved very beneficial.

The number of ports can also be increased, and if properly designed and located at different elevations, the theoretical differential action of the formulas can be substantially exactly obtained. The practical advantage of this procedure is doubtful, however, when it is considered that the coefficient of discharge through the holes is at best only an approximation, and, if the value of c should turn out to be other than expected, the refinements might result in an actual detriment to differential action.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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CINDER CONCRETE FLOORS.

Discussion.*

By Messrs. Edgar Marburg and Guy B. Waite.†

Mr. Marburg.

Edgar Marburg, M. Am. Soc. C. E. (by letter).—In the writer's judgment, the validity of the author's contention, that a design in which cinder concrete is utilized should be governed by rational formulas and prescribed working stresses for the materials involved, cannot successfully be brought into question, provided the design itself is of such a character as to admit of satisfactory analysis, and the requisite empiric data for the unit stresses are available. In the consideration of this important, though essentially simple matter, the discussion should not be befogged by the introduction of irrelevant issues, such as the admittedly variable character of the material and the unsatisfactory conditions as to inspection by which its use is frequently, if not usually, attended. Inadequate inspection cannot reasonably be urged in support of adherence to unscientific methods in relation to the use of this material any more than in relation to any other material. Scientific and therefore economic design is, broadly speaking, impossible in any field of construction without rigid inspection in its execution. If the actual loads on building floors approximated more nearly to those prescribed by building codes, the many failures that would probably have resulted would long ago have enforced the lesson of adequate inspection. That circumstance, however, has no bearing whatever on the merits of the basic question under discussion. It may be, and probably is, true, that cinder, no matter how carefully inspected, is a much more variable product than stone or gravel. If so, that circumstance should be recognized by the selection of correspondingly conservative working

*Continued from August, 1914, *Proceedings*.

†Author's closure.

Mr. Marburg.

stresses in accordance with a principle as old as engineering itself. Such working stresses having been fixed, designs susceptive of rational analysis should be based on suitable formulas and not on tests, except in so far as tests may be made to establish the reliability of the premises —both as to theory and working stresses—governing the design, so that these may then be applied with confidence to other designs of the same general character, but differing as to dimensions, and as to the relative quantity and distribution of the material. To depend wholly on tests, under the conditions stated, would mean a reversion to empiricism unworthy of serious consideration in modern engineering. This statement is just as applicable to cinder concrete as to any other material of construction.

If, on the other hand, the proposed design of which cinder concrete is to be an element is not demonstrably susceptive of rational analysis, only two courses are open: either to prohibit its use, or to have recourse to tests. That statement, too, is just as applicable, however, to stone or gravel concrete, or to any other material, as to cinder concrete. An attempt at differentiation in this connection at the expense of cinder concrete is so manifestly unwarranted that arguments to the contrary would seem superfluous. If such indeterminate designs are to be used, precisely the same precautionary considerations are applicable to cinder concrete as to any other material used under such conditions, namely, that the same care as to the selection of material, workmanship, and inspection should be applied to the construction itself as was applied to the particular elements subjected to test. Furthermore, the information yielded by such tests cannot safely be applied to other similar designs varying materially from those under which the tests were made, as to general dimensions and relative quantities and distribution of material,—that being the well-recognized limitation of empiric, as distinguished from rational methods.

To sum up, at the risk of reiteration, there is, in the writer's judgment, no inherent ground for making any distinctions whatsoever between the general considerations governing the design of reinforced concrete, whether the aggregate be stone, gravel, slag, or cinder. Whether the use of cinder concrete should be permitted where the strength element is an important factor; whether the prescription of proper working stresses would tend to militate against the economic use of this material; and whether its use, at least under certain conditions, should be avoided by reason of the tendency of the embedded steel to corrode, are questions with which the writer is not at present concerned. These questions have no bearing, however, on the fundamental principles which he has endeavored to emphasize, and their discussion in that connection would only tend to becloud the author's basic contention, which, in the writer's judgment, is incontrovertible.

GUY B. WAITE, M. AM. SOC. C. E. (by letter).—The purpose of this paper is: Mr. Waite.

1.—To show that cinder concrete is being used extensively in engineering construction;

2.—To present the fact that, in many cases, this material is not being used in accordance with well-defined engineering principles; and

3.—To obtain discussion among engineers with a view of either having its use included in the category of engineering, or of finding the reason that it should not be so placed.

So far as the writer is able to comprehend the discussions, nothing has been produced to give reasons for not designing cinder concrete according to unit stresses, as required of stone concrete.

The writer agrees with Mr. Marburg that no successful argument can be made for the arbitrary use of reinforced cinder concrete. His position, as head of an institution of engineering, and his connection with the American Society for Testing Materials, entitles him to recognition as an authority on this subject.

The only substitute for unit stresses suggested by any who have discussed the paper was a recommendation for more extensive testing. Of course, with sufficient testing to determine the proper working stresses, it would not be engineering to continue to test for every possible variation in spans, thickness of slabs, and difference in loadings.

Mr. Strehan makes several criticisms, among which are the following:

1.—That "the writer fails to find any plausible basis in the paper on which to establish constants for calculation and unit stresses for design;"

2.—That tension in the cinder concrete is omitted;

3.—That the difference between the conditions of cinder concrete between steel beams and the monolithic condition of all the reinforced concrete is not shown by the writer;

4.—That the trussing effect of side arches on the carrying capacity of any particular arch was overlooked in the paper; and

5.—That the continuous action of reinforcement anchored over the steel beams was ridiculed.

The matters referred to in Criticisms 3, 4, and 5 were mentioned on pages 894 and 899,* and were evidently overlooked by Mr. Strehan. This was the explanation given for the great loads carried by the test arches.

* *Proceedings*, Am. Soc. C. E., April, 1914.

Mr. Waite.

In answer to Criticism 1, the writer would refer to Table 2* giving the unit stresses adopted for this material by several large cities which have placed it on an engineering basis. There is also presented the first of a series of tests made by the Bureau of Buildings of New York City and Columbia University. Further, the writer has his own views regarding its proper working stresses; however, it was not the purpose of the paper to open up side issues for critics to utilize in misleading readers from the real purpose, namely, the establishment of some kind of unit stresses.

In his official connection with the Bureau of Buildings of New York City, Mr. Strehan took part in the tests recorded in Table 2. He refers in his discussion to a coming paper in which he will probably give more up-to-date information on the properties of cinder concrete than is now available. Therefore, it should become his duty as a public official—not the duty of the writer—to assign the exact proper units to be used for New York City. The writer's paper was on the general subject, and the tabulations cover almost every possible assumption.

As to Criticism 2, the writer pleads guilty of the crime of failing to consider tension in the concrete of the reinforced cinder concrete slabs. The report of the Special Committee on Concrete and Reinforced Concrete, which was referred to on pages 913 and 914,* does not recognize tension, even in stone concrete. The stresses developed by the approved loads on cinder concrete, shown on those pages, make such an allowance (or even the talk about it) ridiculous.

As to Criticism 3, it was pointed out that the cinder concrete between steel beams was substantially cut in two and could not act in the certain and continuous manner of monolithic reinforced stone concrete; and for that reason, if for no other, greater stresses should not be allowed on the former than on the latter.

Referring to Criticism 4, Mr. Strehan admits that the outside spans of floor-slabs have not the same capacity as interior slabs, that is, those which have adjoining construction on both sides. As mentioned in the paper, the adjoining slabs thrust against the interior slab. Now, in the test slabs of the constructions discussed in the paper, Figs. 3 to 7,* there were adjoining slabs; but in a large proportion of the slabs in buildings, one side (the outer side) has no slab adjoining to act against thrust.

In discussing Criticism 5, the writer must refer to experiences he has had with light reinforcing members used in the same manner as the wire mesh for the purpose of securing a continuous reinforcement. In 1900 he began the erection of reinforced concrete. His first contract was for cinder concrete floor-slabs located (as generally used in hotel and apartment-house construction) on the lower flanges of

* *Proceedings*, Am. Soc. C. E., April, 1914.

the steel supporting beams. The reinforcement consisted of small thin steel bands drawn continuously over the steel beams and bent down in the central part to reinforce the bottom of the concrete slabs. The metal was kept straight and tight, with the object of securing a continuous action. It was found that in this condition the tamping of the concrete jarred the reinforcement and jerked it in the adjoining slabs to such an extent that the bond was disturbed and cracks appeared in the concrete over each piece of metal. Mr. Waite.

After trying out the system thoroughly, it was abandoned because no practical way of making it continuous could be found. The manner in which the wire mesh and the light reinforcement have been and are being used for the foregoing conditions may be described as follows: The wire or other metal is drawn over the steel beams and then stamped down on both sides and left with sufficient slack to prevent the jerking of the reinforcement, previously mentioned.

An examination of the work actually done in buildings will show much of the reinforcing wire, referred to by Mr. Strehan as being anchored over the beams, simply lying loose on top of them. In this condition there cannot possibly be any continuous action, because the strain comes on the center of the slab before the metal over the supports is drawn tight. All that can be expected of the reinforcement anchored over the beams, is to hold the slab suspended after it has broken in the center. Of course, in a single test slab, sufficient time and pains can be taken to insure continuous action and obtain illusionary results.

Mr. Strehan does not agree with the writer that one kind of reinforcement is as good as another. The writer would modify this statement, to withstand criticism, by saying that what was really meant was that, any forms of steel sufficiently embedded in concrete for the adhesion to develop the full section of the metal will give substantially the same results. This fact was shown for stone concrete in tests by A. N. Talbot, M. Am. Soc. C. E., several years ago, and the tests in Table 4* for cinder concrete seem to show the same result.

The writer referred to the subject of the capacities of various kinds of reinforcement because the arbitrary manner of using cinder concrete has led to all kinds of misrepresentations by sellers of reinforcements. Some have obtained approvals with much less reinforcement than others; all make claims of some peculiarity in their form or kind of material, which makes it superior to all others. In the proposed Building Codes of 1912 and 1913 of New York City (Table 1),* this recognition of superiority was to have been specified in the law.

On account of the seeming protection, by the arbitrary use of the construction, large industries have been built up. It is the writer's

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Mr. Waite. belief that some of these industries have been the father of many of the agitations for new building codes.

Several who discussed the paper referred to the corrosion of steel embedded in cinder concrete. Some seem to think cinder concrete should be restricted, because there is no certainty that the steel is protected against corrosion. As the situation stands, these arguments for restriction are too late; cinder concrete is already used extensively, and the object of this paper is to have it used in a rational manner.

The subject of the corrosion of steel, however, is vital, and should be better understood. It is the writer's experience that the average quality of hard coal cinders mixed in the proportion of 1:2:5, made wet, and tamped thoroughly, will sufficiently coat the surface of the steel with neat cement wash to protect it thoroughly and permanently from corrosion. When the grade of cinders runs too coarse to fill the voids, the concrete is porous, in which case, unless the steel is first coated with cement wash, the concrete is likely to contain air holes extending to the bare steel.

Mr. Himmelwright regrets the fact that the writer did not include the discussion of other forms of cinder concrete than the ordinary plain reinforced construction. The construction discussed in the paper is that which has automatically displaced those referred to by Mr. Himmelwright, and—perhaps it is not exaggeration to say—is the form of construction in universal use, where competition is allowed. The writer may lament this as much as Mr. Himmelwright, but, knowing the facts, he did not wish to confuse the subject by the introduction of constructions so far removed in relationship that it would be useless to attempt to make a formula to include them all.

In defence of the writer's rational treatment of the subject and of the general tabulations, to cover the use of almost all constants, he begs to state that this was done principally because, in past discussions, before aldermanic committees, etc., the unit-stress idea has been ridiculed, and more than one member of the Society has scouted the idea that a rational treatment was practical.

It is hoped that engineers will awaken to their own interests by insisting that cinder concrete, when used as an engineering construction, shall be elevated from the field of irresponsible contractors, referred to by several in their discussions, to the field of Civil Engineering.

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PAPERS AND DISCUSSIONS

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THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

Discussion.*

By J. A. L. Waddell, M. Am. Soc. C. E.†

Mr. Waddell.

J. A. L. Waddell, M. Am. Soc. C. E. (by letter).—The writer's hearty thanks are herewith extended to all who have done him the honor to discuss this paper, and he is specially gratified by the appreciative reception which it has met at their hands.

In this résumé he will treat only of those topics on which a disagreement of opinion has been expressed, or those which afford data for a further development of the subject. Although many of the points raised in several of the discussions are foreign to the subject at issue, dealing with bridge designing and construction in general, nevertheless, the writer has answered such points to the best of his ability and according to his experience.

Replying to the suggestion by Mr. Le Conte, which has been offered also by Mr. Molitor, that experiments be made with aluminum as an alloy material for bridge steel, the writer regrets that hitherto he has been unable to collect any authentic information concerning aluminum-steel alloys; nor did he pay much attention to the possibilities of such a combination on account of the previous high price of aluminum. With a present price of only 20 cents per lb., however, and the possibility of attaining an elastic limit of 100 000 lb. per sq. in., the conditions are changed materially, making the expenditure of much time, effort, and money on the necessary investigation well worth while. Mr. Le Conte stipulates that before adding the cake of aluminum to the molten steel, the latter must first be "'thoroughly purified' and its elastic limit raised thereby to 60 000 lb. per sq. in." How this is to be done, he does not say. If steel, high in carbon,

* Continued from September, 1914, *Proceedings*.
† Author's closure.

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produced by the electro-metallurgical process, were used, such a great elastic limit might possibly be attained; but, judging from what was told the writer by some gentlemen interested in titanium, the use of that metal as a scavenger does not materially raise the elastic limit or the ultimate strength of the steel, its effect being mainly to make the product more uniform and reliable.

It is evident that much experimenting would have to be done in order to determine how best to produce an aluminum-steel alloy suitable for bridgework.

Mr. Le Conte's estimate of 2.5 cents per lb. for the excess cost of the manufactured alloy appears to be liberal enough; for purified steel smelted by the electro-metallurgical process, as claimed by its French producers, costs only 0.9 cent per lb. more than carbon steel, which amount added to the cost of the aluminum, would make the excess only 1.5 cents per lb., leaving 1 cent per lb. to cover a possible slightly increased cost of rolling the plates and shapes and the extra profit which, doubtless, would be insisted on by the manufacturers on the plea of expense due to innovation from current practice.

All that Mr. Petinot has said in his discussion concerning the use of titanium in the manufacture of steel is of the greatest interest and value to the Engineering Profession; but it is to be regretted that he omitted to state a number of facts of importance which he mentioned to the writer during a long conversation on the subject held a day or two previous to the presentation of the paper to the Society. At the moment of writing, the writer cannot recall the omitted information, except only that (as he indicated in the comment on Mr. Le Conte's discussion) titanium purifies and makes much more reliable the steel with which it is alloyed, but does not increase to any great extent the elastic limit or the ultimate strength.

The writer is glad to note the corroboration of the claim made by Mr. T. L. Willson to the effect that it is practicable to manufacture satisfactory nickel steel by the addition of ferro-nickel instead of the pure metal. It seems strange that Mr. Willson has not seen fit to take part in the discussion of this topic; for he has been given several opportunities to do so.

As, according to Mr. Petinot, the cost of purifying steel by the use of titanium amounts to only 25 or, at most, 50 cents per ton, it would seem advisable, after his claims have been well established, for the Profession to insist on its use in the manufacture of all carbon steel for bridgework.

The writer desires to emphasize forcibly the remarks of Mr. Moisseiff concerning the necessity of proportioning all highway bridge floors so as to carry properly the heavy concentrated live loads imposed on them by automobile trucks, and concerning the inefficiency of buckle-plates to support pavements subject to such loads. Every

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bridge designer should take these statements to heart, and be guided thereby in detailing his floors for highway structures; for, no matter how remote or insignificant a highway bridge may be, it is within the realm of possibility that at any time it may have to carry heavy automobile trucks.

Concerning Mr. Moisseiff's warning about the greater distortions (and, therefore, greater secondary stresses) in bridge members when high-alloy steel is used: This is evidently well founded; but, on the other hand, the fact that the members are more slender in an alloy-steel bridge than in a corresponding carbon-steel structure offsets somewhat the injurious effects of the extra distortion. Moreover, in the best modern bridge engineering practice, the greater part of the maximum secondary stress on each main member is overcome by forcing a reverse secondary stress into the metal during erection.

Concerning the efficiency of nickel-steel rivets, the writer would refer Mr. Moisseiff to the discussions by Messrs. Hodge and Skinner at the meeting of the Society when the paper was presented, only a part of which has been recorded in print in Mr. Skinner's discussion. It seems from these discussions that even very long and large nickel-steel rivets can be driven satisfactorily in both shop and field. In alloy-steel bridges, as in carbon-steel bridges, it will, necessarily, be the practice to use much softer steel for rivets than for plates and shapes in order to make possible the cutting out of defective rivets.

Mr. Skinner's question as to the possibility of developing a super-strong alloy steel by subjecting finished members to heavy test loads at the shops, opens up a new subject. The writer thinks that such a practice might be feasible for eye-bars, in which the holes might be drilled small for testing and be enlarged after the bars are stretched; but that kind of manipulation could not well be applied in the case of compression members, unless they are all pin-connected, which construction, although possible, would involve many complications and difficulties in detailing.

Mr. Molitor's remarks concerning piping and segregation are both pertinent and timely; and it cannot be questioned that the manufacture of "high-alloy" steels for bridges should receive far better attention than that of the commercial carbon steel for such structures. Manufacturers object to any requirement that tends to prevent them from turning out their product quickly and in great quantities. One cannot blame them for this, because it is human nature to desire to make as great a profit on work as possible; but if engineers would insist on obtaining what they know to be necessary in order to secure the best results, and if they would make their clients pay a proper price for the improvement in manufacture, it should not be very difficult—at least not impossible—to effect important innovations in the art of bridge building.

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It is this reluctance of the metal manufacturers to adopt innovations and to take special pains, which has prevented the Engineering Profession from having had at its disposal during the last five or six years a nickel steel for bridge building with a minimum elastic limit of 60 000 lb. per sq. in. A little extra care in the furnace manipulation and a moderate increase in the cost of shopwork are all that is necessary to provide such an alloy in as large quantities as may be desired; and the excess pound price for the finished material should only be in direct proportion to the actual excess cost of preparing the rolled sections and of manufacturing them into bridges.

In reference to the theory of segregation and piping, it is to be hoped that engineers will insist on metal manufacturers giving the "Liquid Forged Steel Process" a fair trial. If it be true that the use of this process by purifying the steel and by avoiding piping will raise the elastic limit and ultimate strength of the metal 20%, a most important advance can be made by its adoption in the attainment of a true high-alloy steel for bridge building.

The writer will have to take issue with Sir Bradford Leslie concerning the effect of impact on bridges. Sir Bradford appears not to believe that impact from rapidly moving live loads on bridges is an actuality. The writer knows that it is, for he has both seen and measured it on a number of occasions. In one series of experiments on spans as a whole (not on their individual members), he found increased deflections from motion varying from 20% for spans of 160 ft. to 50% for a span of 88 ft. It is true that the longer spans were pin-connected and of ancient design, and that the shortest one was of the riveted type and disgracefully detailed; nevertheless, other experiments of his on properly designed bridges all show more or less increased deflections due to motion. Again, a committee of the American Railway Engineering and Maintenance of Way Association has made many hundreds of determinations of impact on bridges; and its results are a matter of record.

Sir Bradford questions the writer's reason for assuming that long-span railway bridges will have, at least, two tracks. He would, indeed, be a short-sighted American engineer who, to-day, would counsel his client to build a bridge of very long span with a single track; although it would be quite legitimate to adopt the writer's patented expedient of designing it in such a manner that, in the future, duplicate trusses might be added so as to double its carrying capacity.

It is not a dread of wind pressure, as Sir Bradford supposes, which causes American bridge engineers to double-track their long-span bridges (for the width between trusses can be increased to any desired amount without providing an additional track), but because they recognize the fact that if the client can afford to spend a large sum of money for a railroad bridge, the traffic of the line should be great

enough to call for a double track, if not at present, at any rate in the near future. Mr. Waddell.

Mr. Fergusson's discussion is really based on the writer's former paper, "Nickel Steel for Bridges",* as it hardly deals at all with the question of using high-alloy steels. Moreover, Mr. Fergusson is apparently somewhat behind the times; for, in several instances, he questions established facts.

He says that "the use of nickel steel cannot be adopted, generally, as the best material for all classes of bridgework." All that prevents this to-day is the unwillingness of the manufacturers of bridges to adopt the innovation and to put a reasonable price on the product.

He says, also, that "carbon steels, for a tensile strength of 40 tons per sq. in., have advantages over nickel steels carrying low percentages of nickel." Any ordinary carbon steel having a minimum ultimate strength of 40 tons per sq. in. would be too brittle to use in bridge construction. It is true that, by taking special pains in its manufacture and drilling solid all the rivet holes, it might be practicable to use such high carbon steel; but it would be bad policy to do so when a nickel steel some 40 or 50% stronger is obtainable at a not entirely unreasonable price. Nickel steel having an ultimate tensile strength of 60 tons is more uniform and reliable than an ordinary carbon steel of 40 tons—at least, that is the writer's opinion, based on an extended experience with bridge metals.

Mr. Fergusson says "the testing of nickel steel during the process of manufacture is very complicated, the material is subject to great variations;" etc.

Such is not the writer's experience in the building of nickel-steel bridges, nor, unless he is greatly mistaken, is it that of Mr. Hodge or of other American engineers who have had occasion to build structures with that alloy.

Mr. Fergusson says:

"To use a nickel steel of very high tensile strength, chiefly to save weight in the bridge material—unless where the conditions are imperative—would seem to be extravagant, and even unwise. There must be risk in trusting to the highest limit of tensile strength of a manufactured steel which is known to be very variable."

In answer to this the writer would state that nickel steel is not "very variable", as can be seen by a systematic study of his inspectors' reports, which are published in Appendix A of the paper on "Nickel Steel for Bridges." Those records show clearly that the different kinds of nickel steel are just as regular in their elastic limits and ultimate strengths as the different kinds of carbon steel.

There is no more risk in "trusting to the highest limit of tensile strength" of nickel steel than in trusting to the highest limit of

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

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tensile strength of carbon steel—moreover, it is not the highest limit which is adopted as a minimum in specifications, but one fairly close to the lowest limit, so chosen as to avoid the necessity for many rejections.

The writer wonders what Mr. Fergusson deems is the object of bridge engineers in ever using nickel steel for their structures, unless it be "to save weight in the bridge material." That is the sole *raison d'être* for its adoption.

Mr. Fergusson says: "The proportion of 3.5% of nickel, which he [the writer] gives, appears to be too low." For plate-and-shape steel this is as much as it is either profitable or advisable to use. Any greater percentage would add to the expense of the rolled metal and increase the cost of the shopwork. However, for eye-bars in which the amount of shopwork is small, it is economical to increase the percentage of nickel to about 4.25.

Mr. Fergusson says: "As he [the writer] does not mention carbon in his analysis, it may be presumed that he has eliminated it; surely this is a difficult operation in commercial manufacture?"

It certainly would be—but, unfortunately for Mr. Fergusson's deduction, the writer did not attempt to cut out the carbon—far from it. The great strength of nickel steel in bridgework is largely due to the fact that the addition of nickel permits of a substantial increase in the percentage of carbon. The nickel itself adds to the strength of the metal, but at the expenditure of considerable money; while the increase in the carbon content adds to the said strength without augmenting the cost at all. Surely Mr. Fergusson has failed to read with any care the paper on "Nickel Steel for Bridges"; for if he had done so, he would not have made such a blunder. The specifications for the various classes of nickel steel given in Table 4 of that paper call for the following compositions:

Ingredients.	Percentages.		
	Rivet steel.	Plate-and-shape steel.	Eye-bar steel.
Nickel	3.50 (3.25 to 3.75).	3.50 (3.25 to 3.75).	4.25 (4.00 to 4.50).
Carbon	0.15 (0.12 to 0.18).	0.38 (0.34 to 0.42).	0.45 (0.40 to 0.50).
Phosphorus	0.03 maximum.	0.03 maximum.	0.03 maximum.
Sulphur	0.04 maximum.	0.04 maximum.	0.04 maximum.
Silicon	0.04 maximum.	0.04 maximum.	0.04 maximum.
Manganese	0.6 (0.55 to 0.65).	0.70 (0.65 to 0.75).	0.80 (0.75 to 0.85).

Speaking of the strengths of nickel steels of varying percentages of nickel, Mr. Fergusson states that "the increase of nickel requires a decrease of carbon, together with an increase of manganese, otherwise, although the breaking load may be the same, the value of the elastic limit will fall rapidly." Within the limit of the writer's experiments,

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which do not cover alloys higher in nickel that 4.5%, he has found that just the opposite condition exists; for increasing the carbon content raises both the elastic limit and the ultimate strength.

The writer does not claim that nickel steel having a greater percentage of nickel than 3.5 is absolutely unworkable in the shops; but that commercially it is so. On page 103 of the paper on "Nickel Steel for Bridges" will be found the following:

"Curiously enough, the superior limit of nickel steel for workability in bridge shops varies but little, if any, from the economic amount based on present prices of steel and nickel. For plates and shapes, which form the principal part of bridge superstructures, any materially greater percentage than 3½ renders the alloy too refractory for the various shop manipulations to which it would be subjected in manufacturing it into bridges, although superstructures built of steel still higher in nickel would be perfectly safe and satisfactory for operation."

If Mr. Fergusson will read the results of the investigations of Messrs. Carpenter, Hadfield, and Longmuir,* he will see the danger of using high percentages of nickel in bridge steel; for these gentlemen have demonstrated the existence of a "brittle zone" in nickel steels, beginning with a percentage of nickel lying between 4.25 and 5.0, and ending at 20. These investigations were referred to in the writer's former paper.

Mr. Fergusson points out what he claims to be "two disadvantages in nickel steel for bridgework:

"(*a*) The difficulty of welding, and working;
"(*b*) The necessity of keeping the carbon extremely low—under 0.20%—whereas the fatigue resistance of steel is increased enormously by raising the percentage of carbon."

To these criticisms the writer would reply as follows:

First.—Welding in bridgework is now a thing of the past, and has been ever since the days of wrought-iron bridges.

Second.—There is certainly more difficulty in manipulating nickel steel in the shops than in handling carbon steel there, but the extra cost is trifling, amounting to between 0.15 and 0.25 cent per lb. This is a bagatelle compared with the advantages gained by the use of the alloy.

Third.—There is no reason whatsoever for "keeping the carbon extremely low."

Fourth.—The fatigue of steel in bridges is now conceded by the leading American engineers to be entirely a myth. For many years it was a great bugbear to bridge builders; but to-day it is a well-known fact that there is no "fatigue" of metal unless the elastic limit is exceeded; and most of the reputable American bridge engineers

* *The Engineer* (London), November 24th, 1905.

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aim to keep the actual intensities of working stresses in their structures down to about one-half of the elastic limit.

Answering Mr. de Broë's discussion, in reference to the price of steel: the writer assumed in 1907 that the average pound price for plate-girder spans erected was 4.0 cents; and to-day it is half a cent less, or 3.5 cents. Subtracting from this, as an average, 1.25 cents for freight and erection, would make the pound price, f. o. b. cars at the shops, 2.25 cents, which is about right. During bad times in the United States plate-girder spans can be bought for less than 2.0 cents per lb., f. o. b. cars at the works; but when times are very good, this figure is increased some 50% or more. Mr. de Broë's quotation of £13/10/0 per ton, as the highest price paid in England during the last few years for bridge superstructures, corresponds to nearly 3.0 cents per lb. The writer does not think that bridges can be manufactured in England as cheaply as they can in the United States, but admits that in this opinion he may be mistaken.

In regard to the question of impact brought up by Mr. de Broë, the writer has treated that subject already in his reply to Sir Bradford Leslie's discussion; but he would remark that although the recorded impact on that 40-ft. span at 45 miles per hour may truly have been zero, a lower velocity of train might readily have indicated a high percentage—such, at least, has been the experience of American engineers (among others the writer) in testing bridges for deflection under static and moving loads. The writer is quite convinced from a practical experience in the designing, building, examination, and testing of bridges, extending over a period of more than 33 years, that the use of proper impact formulas or curves is a necessity in truly scientific design.

He takes pleasure in complying with Mr. de Broë's request for "the more accurate formula" which he has evolved from the published tests of a commission of the American Railway Engineering and Maintenance of Way Association appointed to experiment on impact from moving loads on railroad bridges. The formula for coefficients is

$$I = \frac{165}{L + 150};$$

or, in terms of percentages, it is

$$I = \frac{16\,500}{L + 150},$$

where L is the length of span, in feet, covered by the moving load. The highest of the three curves in Fig. 28, prepared from this formula, gives the coefficient of impact for all spans between zero and 1 000 ft. At these limits, the coefficients are, respectively, 1.10 and 0.14.

The writer regrets that he cannot at the present writing give Mr. de Broë the name of the publication containing the results of

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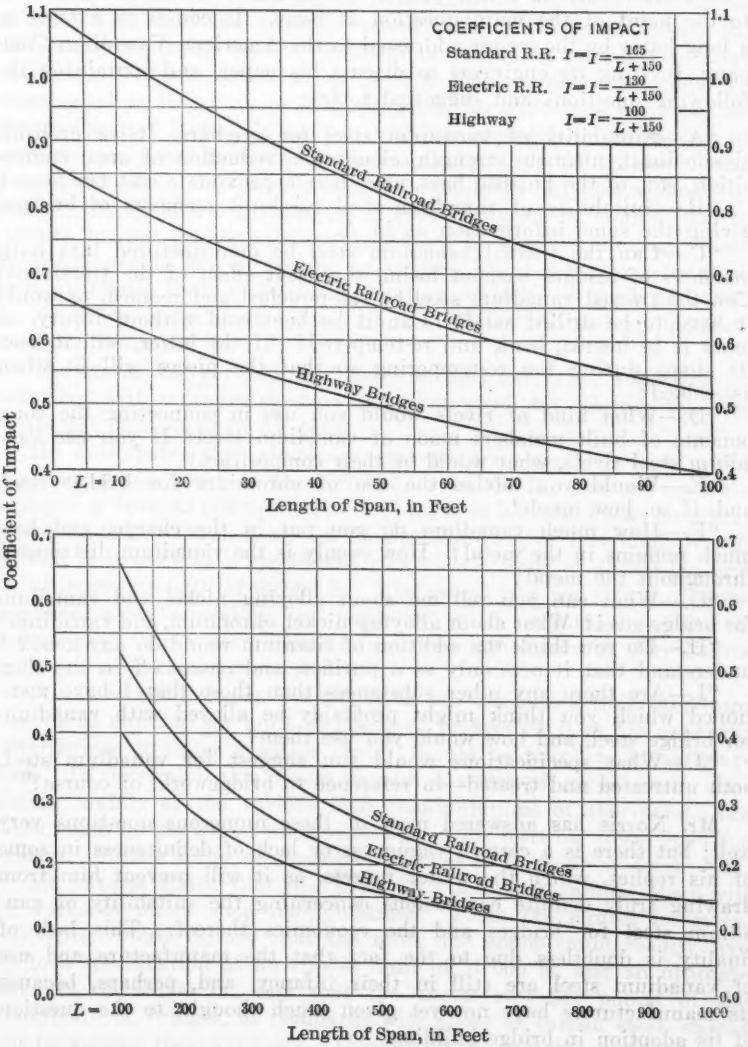

FIG. 28.

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the impact experiments from which the foregoing formula was prepared.

The discussion by Mr. Norris is exceedingly valuable, being directly to the point of the main question at issue. It comes as a reply to a long letter by the writer addressed to the American Vanadium Company, inviting its engineers to discuss his paper, and containing the following questions and suggested topics:

"A.—Suitability of vanadium steel for eye-bars. Give probable elastic limit, ultimate strength, elongation, reduction of area, composition, etc., of the finished bars, also their approximate cost per pound.

"B.—Suitability of vanadium steel for built members of bridges, giving the same information as in A.

"C.—Can the treated vanadium steel be manufactured into built members of bridges without losing the great effect of the treatment? Can the treated vanadium steel be sub-punched and reamed, or would it have to be drilled solid? Can it be bent cold without injury, or must it be heated, bent, and re-tempered? If the latter, will it keep its shape during the re-tempering so that the pieces will fit when assembled?

"D.—What kind of rivets would you use in connecting the components of built members made of vanadium steel? If you use vanadium steel rivets, what would be their composition?

"E.—Would you advise the use of chromium for bridge steel, and, if so, how much?

"F.—How much vanadium do you put in the charge, and how much remains in the metal? How evenly is the vanadium distributed throughout the metal?

"G.—What can you tell me about alloying nickel and vanadium for bridge steel? What about alloying nickel, chromium, and vanadium?

"H.—Do you think the addition of titanium would do any good? I understand that it acts only as a purifier, and passes off in the slag.

"I.—Are there any other substances than those that I have mentioned which you think might profitably be alloyed with vanadium for bridge steel, and how would you use them?

"J.—What specifications would you suggest for vanadium steel, both untreated and treated—in reference to bridgework, of course?"

Mr. Norris has answered most of these numerous questions very well; but there is a certain vagueness or lack of definiteness in some of his replies, which the writer regrets, as it will prevent him from drawing truly definite conclusions concerning the suitability of vanadium steel for bridges and the economics thereof. This lack of finality is doubtless due to the fact that the manufacture and use of vanadium steel are still in their infancy, and, perhaps, because its manufacturers have not yet given much thought to the question of its adoption in bridge building.

It will be well to take up each of the preceding points and questions consecutively, and see what has been answered and what remains to be determined.

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A.—It is evident from the discussion that vanadium steel is eminently fitted for the manufacture of eye-bars. The elastic limit for full-sized, chrome-nickel-vanadium bars varied in the tests from 63 280 to 80 480 lb., and the ultimate strength from 93 500 to 99 800 lb., the results depending on the drawback or annealing temperature after quenching. It seems logical to assume that no difficulty would be experienced in living up to a specification for such bars requiring an elastic limit of 70 000 lb. and an ultimate strength of 95 000 lb., or perhaps even 5 000 lb. more for the elastic limit, as the specimen tests gave 83 000 lb. Mr. Norris suggests specifying an elastic limit of from 65 000 to 80 000 lb., and an ultimate strength of from 85 000 to 105 000 lb.; but bridge builders must work to specified minima, hence the writer would like to assume, in writing his specifications for such bars, an elastic limit of 70 000 lb., and an ultimate strength of 95 000 lb., or, if practicable, an elastic limit of 75 000 lb. and an ultimate strength of 100 000 lb.

The excess cost per pound of these finished bars after annealing, as compared with ordinary eye-bars of carbon steel, Mr. Norris indicates would not be more than 3.5 cents.

He anticipates that simple carbon-vanadium-steel eye-bars would have an elastic limit of from 60 000 to 75 000 lb., and an ultimate strength of from 85 000 to 100 000 lb.; but, as it is evident that no actual experiments have been made on finished eye-bars of this kind, it will be well to assume as a specification an elastic limit of 65 000 lb., and an ultimate strength of 90 000 lb. The excess cost per pound of these eye-bars, after annealing, as compared with ordinary eye-bars of carbon steel, the writer deduces from two statements of Mr. Norris to be not more than 1.5 cents.

B.—It is not quite so evident from the discussion that vanadium steel is as suitable for built members of bridges as it is for eye-bars; nevertheless, it seems probable that it would be found to be satisfactory. To determine this would require a series of experiments, consisting mainly of the various shop manipulations of the metal, like those made by the writer some years ago in his tests on nickel steel for bridges. Such a series of experiments could be made at moderate cost.

Mr. Norris states that his Type "A" chrome-vanadium steel will be workable under shop manipulations. From the figures he gives, the writer would deduce that the mills and shops could readily live up to a specification of an elastic limit of 70 000 lb. and an ultimate strength of 95 000 lb., and that the excess cost per pound of the manufactured bridges, as compared with those of carbon steel, should not be greater than 4.0 cents. This would include the cost of annealing (which treatment, it is fair to assume from the general tenor of the discussion, would be obligatory), the extra cost of shopwork, and a small allowance for contingencies.

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From what Mr. Norris anticipates concerning the use of simple carbon-vanadium steel for built members, one may conclude that it would be safe to specify an elastic limit of 60 000 lb., and an ultimate strength of 90 000 lb., and to assume an excess cost per pound of the manufactured bridges, as compared with those of carbon steel, not greater than 2 cents. This last figure is derived thus:

Excess cost of raw material	1.0	cent per lb.
Heat treatment	0.5	" " "
Excess cost of shopwork	0.25	" " "
Contingencies	0.25	" " "
Total	2.0	cents per lb.

C.—It appears likely, from what Mr. Norris remarks, that treated vanadium steel can be manufactured into built members of bridges without losing the great effect of the treatment; but a few practical experiments would determine this beyond the peradventure of a doubt. It is likely that it would prove better to drill the rivet holes than to sub-punch and ream them. This would add only a small amount to the cost per pound of the shopwork.

Mr. Norris does not say whether vanadium steel can be bent cold without injury; hence one may surmise that it cannot. However, this would not militate greatly against its use in bridgework, because in long-span structures (the only kind now under consideration) there should be very little, if any, metal to be bent.

D.—In respect to rivets, Mr. Norris advises for chrome-vanadium steel an elastic limit of from 50 000 to 65 000 lb., and an ultimate strength of from 70 000 to 90 000 lb.; and for simple carbon-vanadium steel an elastic limit of from 40 000 to 55 000 lb., and an ultimate strength of from 65 000 to 85 000 lb. He says "There should be no difficulty in driving rivets of either of these types of steel." True enough; but there might be great difficulty in cutting them out—and, alas, owing to the frailty of human nature, it is impossible to build bridges without having to remove at least a few rivets—and generally a good many of them.

In his experiments on nickel steel for rivets the writer encountered this difficulty, and he was obliged, in consequence, to lower the carbon content (and hence also the elastic limit and the ultimate strength) in his specifications for rivet nickel steel. Nothing but some thorough experiments on vanadium-steel rivets of varying compositions will determine how high a steel it will be safe to use for such rivets.

Concerning rivets for bridges built of high-alloy steels, it is a foregone conclusion that the ratio of the strength of the alloy-steel rivets to that of carbon-steel rivets cannot be as great as the corresponding ratio of strength of plate-and-shape alloy steel to that of

plate-and-shape carbon steel. On this account, in high-alloy steel bridges it will be necessary to use proportionately either more rivets or greater rivet diameters—or both. Mr. Waddell

E.—Mr. Norris has spoken in favor of the use of chromium for vanadium bridge steel, although he indicates a preference for simple vanadium steel. It is possible that the latter is preferable for moderately long spans and the former for very long ones. The economic study based on the preceding deductions from the discussion, which is to follow herein, will determine this point. The quantity of chromium recommended by Mr. Norris varies from 0.6 to 0.9%, in combination with manganese varying from 0.4 to 0.6%, or 0.8 per cent. The writer would be inclined to go a few points, say, 20, higher with the manganese, as his experiments on nickel steel indicated the advantage of such a policy; but, perhaps, the addition of the chromium might necessitate some reduction in the manganese.

F.—For a number of years it was thought by the Engineering Profession in general that vanadium in steel acted only as a scavenger, none of it remaining in the finished product, but all passing off with the slag. However, it seems that such is not the case, but that about 80% of the vanadium which is added to the charge remains in the finished product.

It seems, according to Mr. Norris, that the vanadium is very evenly distributed through the ingot, and that not only is there no danger whatsoever of its segregation, but also that its presence in the molten metal tends to prevent the segregation of other substances—notably carbon. This is most reassuring, and is a strong point in favor of the use of vanadium steel for bridge building.

G.—Mr. Norris does not say much about alloying nickel with vanadium or about combining nickel, chromium, and vanadium. Such combinations are purely a question of economics; and only elaborate investigations can determine what should be the constituents and their proportions to produce the best and most economic alloy. From what Mr. Norris states, one might anticipate that the gain by adding nickel to vanadium-carbon steel or to vanadium-chromium steel would be either very small or non-existent.

H.—Mr. Norris is quite convinced that the addition of titanium to vanadium steel would be useless. Perhaps the explanation for this is that the 20% of vanadium which passes off into the slag acts as a scavenger, thus obviating the necessity for any further purification—which is the sole function of the titanium.

I.—Mr. Norris does not reply to the question as to whether there are any other substances than those herein mentioned, which might profitably be added to vanadium steel. His reason, probably, is that he does not know, never having tried the addition of such elements.

Mr. Waddell. It is only extensive experiments that will furnish an answer to this question.

J.—The specifications for the composition and properties of vanadium-carbon steel and vanadium-chromium steel, suggested by Mr. Norris, are very wide in their range, but they are probably as definite as the present state of the art of manufacturing those alloys will allow.

There is one statement near the beginning of Mr. Norris' discussion to which the author must take exception, namely, "In the case of nickel steel, in order to obtain an elastic limit of 60 000 lb. or more, it is necessary to increase the percentage of carbon to undesirable limits."

The plate-and-shape steel experimented on by the writer in his nickel-steel investigation had an elastic limit slightly in excess of 60 000 lb., and was a perfectly satisfactory metal in every essential particular for bridgework. It is true that its manufacture was harder on the tools than ordinary carbon steel, but that feature simply added slightly to the cost of the shopwork. The metal withstood all the shop manipulations as well as ordinary carbon steel; it was just as reliable in every particular; and there was no danger whatsoever from brittleness.

The economics of Mr. Norris' vanadium-carbon and vanadium-chromium steels, as compared with ordinary carbon steels and with the nickel steels which can be obtained to-day for bridgework at the prices last quoted to the writer by the manufacturers, will now be considered. Using the averages just determined in this résumé, we have for the vanadium-carbon steel an elastic limit of 60 000 lb. and an excess pound price of 2.0 cents as compared with carbon steel; and for the vanadium-chromium steel an elastic limit of 70 000 lb. and an excess pound price of 4.0 cents.

On Figs. 29 and 30 are plotted, for simple-span bridges and for cantilevers, respectively, the weights per linear foot of metal for various spans of carbon steel, vanadium-carbon steel, vanadium-chromium steel, nickel steel for $E = 50\,000$ lb., and nickel steel for $E = 55\,000$ lb.; and on Figs. 31 and 32 are shown, for simple-span bridges and for cantilevers, respectively, the economics of the said five classes of steel considered, as follows:

A.—Carbon steel, $E = 35\,000$ lb.

B.—Nickel steel, $E = 50\,000$ lb., at an excess cost, delivered, of 2 cents per lb.

C.—Nickel steel, $E = 55\,000$ lb., at an excess cost, delivered, of 2.5 cents per lb.

D.—Vanadium-carbon steel, $E = 60\,000$ lb., at an excess cost, delivered, of 2.0 cents per lb.

E.—Vanadium-chromium steel, $E = 70\,000$ lb., at an excess cost, delivered, of 4.0 cents per lb.

Mr. Waddell.

Referring to Fig. 31, it is seen that, for simple-span bridges, under the conditions assumed, vanadium-carbon steel shows, for all span lengths, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins to develop an economy over carbon steel at a span length of about 600 ft., and over the two nickel steels at a span length of about 700 ft.

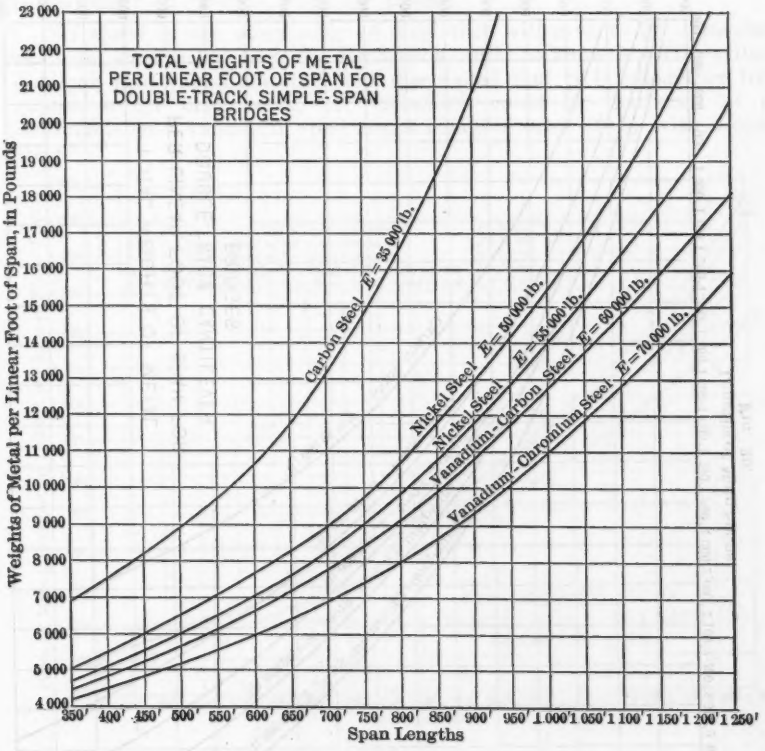

FIG. 29.

Referring to Fig. 32, it is seen that, for cantilever bridges, under the conditions assumed, vanadium-carbon steel shows for all span lengths, as in the case of simple spans, a small but material advantage over all the other steels; and that the vanadium-chromium steel begins to develop an economy over the carbon steel and the nickel steels when the main opening has a length of about 1 600 ft.

Contrary to the writer's anticipation, the lines for vanadium-carbon steel and vanadium-chromium steel, at the excess pound prices assumed, do not cross each other. They would intersect, for cantilever

Mr. Waddell.

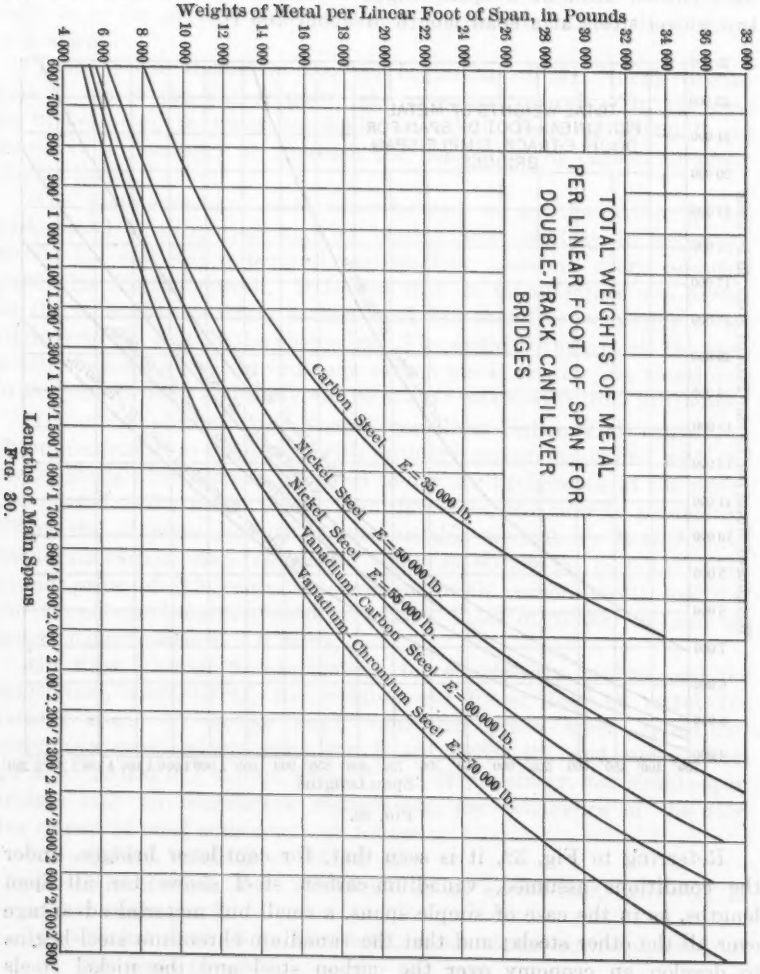

FIG. 30.

bridges, at a main opening of 2 800 ft., which is about the practicable limit for $E = 70\,000$ lb. Mr. Waddell.

Some 8 or 9 years ago certain leading steel manufacturers quoted to the writer an approximate price of 8 cents or more per lb. for vanadium steel bridge superstructures, delivered f. o. b. cars at the shops. It was this which caused him to make the following statement in his paper on "Nickel Steel for Bridges":

"If there is any other alloy of steel that will give as good results as nickel steel at less cost, the Profession ought to know it. The writer did look into vanadium steel, but discovered that it is altogether too expensive for bridgework. If vanadium could be furnished at a reasonable cost, vanadium steel might compete successfully with nickel steel."

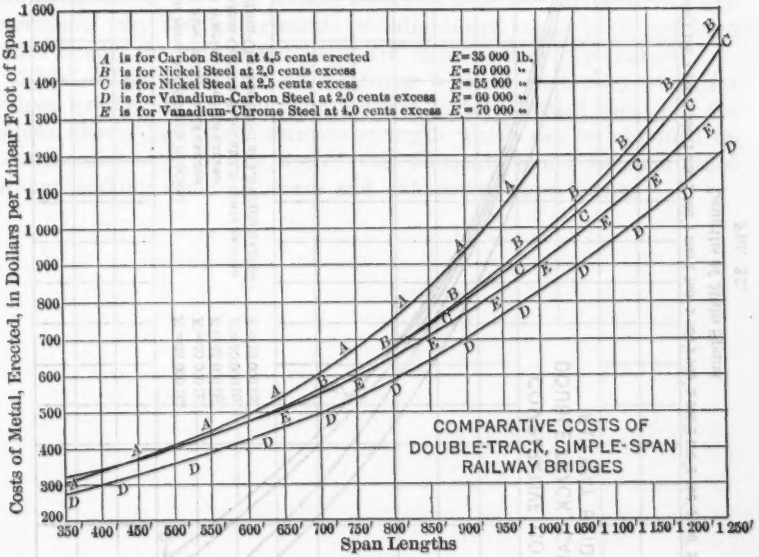

FIG. 31.

Mr. Norris states that "the price of vanadium has been reduced 60% within the past 3 years, and it is now low enough to bring it within consideration for use in steel for eye-bars and other parts of long-span bridges." This statement is corroborated by the preceding economic study; but it must be remembered that the latter is based on somewhat uncertain data. However, the writer aimed to be conservative when assuming the same from the statements in Mr. Norris' discussion; hence, it is easily possible that the vanadium steels, if thoroughly investigated, would show far better economic results than those just determined—in fact, it is not impossible that the ideal

Mr. Waddell.

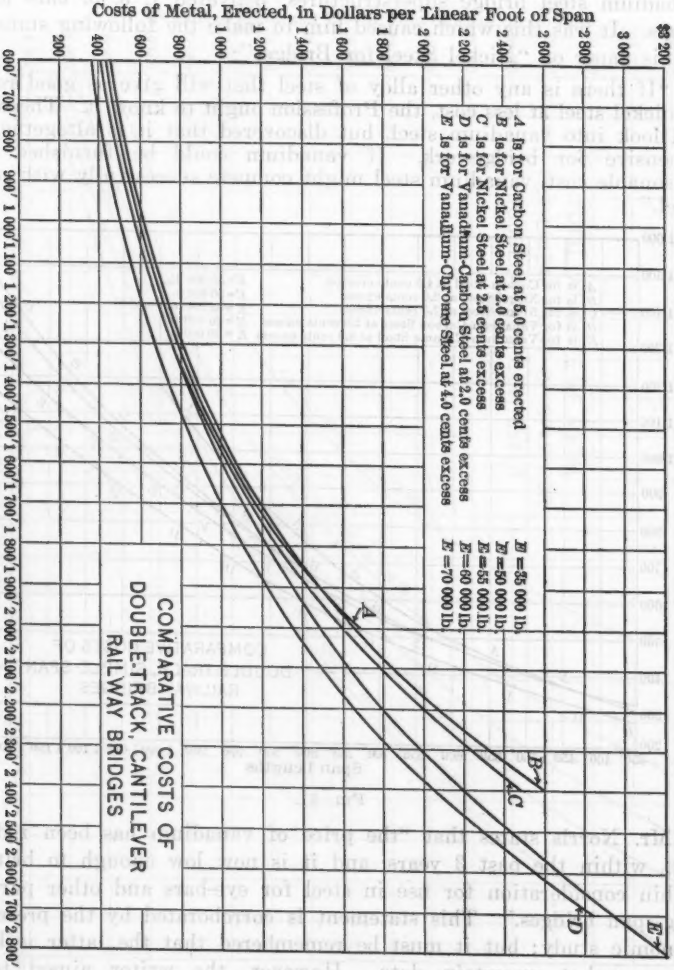

FIG. 32.

workable bridge steel having a minimum elastic limit of 100 000 lb., and of moderate cost, is attainable by the use of vanadium. Mr. Waddell.

Summarizing the findings of the discussion and the résumé, the following conclusions can be drawn:

First.—Titanium as a scavenger of carbon steel promises good and useful results at exceedingly low cost. Although it does not increase greatly the elastic limit or the ultimate strength of the metal, it makes it much more uniform and reliable. On this account it should be used in a few cases on bridgework; and then, if it be found satisfactory, its adoption should be made obligatory by the railroad companies and the other builders of carbon-steel bridges.

Second.—There appear to be great possibilities in the use of aluminum as an alloy for bridge steel; but, as far as the writer can determine, very few experiments in aluminum steels have yet been made; hence, the said possibilities are more or less hypothetical.

Third.—The possibility of obtaining a good, high-alloy steel for bridges by the use of vanadium appears to be a settled fact; but the highest elastic limit and ultimate strength which can be obtained on a commercial basis by the use of that element cannot be determined without making some elaborate and exhaustive experiments.

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THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE.

Discussion.*

By Charles H. Lee, Assoc. M. Am. Soc. C. E.†

Mr. Lee.

Charles H. Lee, Assoc. M. Am. Soc. C. E. (by letter).—Realizing the somewhat local character of his paper, the writer has been highly gratified to find that it has brought forth discussion based on such widely divergent as well as similar experience. He especially appreciates the broad and constructive contributions of Messrs. Smith and Meinzer, whose wide experience with ground-water problems qualifies them to speak with authority.

The writer heartily agrees with Mr. Owen that there is a great need in all sections of the United States for definite scientific knowledge regarding the principles which should govern ground-water development. He would not, however, place such knowledge in the realm of the unattainable, as both Messrs. Owen and Allen in their opening paragraphs seem inclined to do. The Engineering Profession has already made considerable progress in the formulation and application of such principles. In Europe, for instance, where the demand for municipal water supplies has reached the point where the limit of every available source must be known, there has been accumulated such a fund of knowledge derived from investigation and experience that the subject is recognized as a distinct branch of the profession. In the United States the exhaustive investigations of ground-water supply, such as that made on Long Island by the City of New York and the work of the United States Geological Survey, show that

* Continued from September, 1914, *Proceedings*.

† Author's closure.

Mr. Lee.

in America, also, the subject has advanced far beyond the stage of conjecture. It is the writer's opinion that the time is now ripe for the formulation of general principles and methods of investigation and development of ground-water among American engineers, and it was with this in view that his paper was written. It is to be hoped that similar papers will be presented by members of the Society who have had opportunity to study other types of ground-water occurrence, in order that a more complete presentation of the subject in its most recent development may become available to the profession.

The writer was much interested in the statements of both Messrs. Smith and Meinzer that the term "water-tight" as applied to ground-water reservoirs is a relative one, and that, in many arid States, it does not apply at all. Their experience and the writer's more recent observations outside of California are in accord in this matter, and the first conclusion of the paper (page 818*) should be modified accordingly. In all the California basins which have come under the writer's observation, however, the term can be used for all practical purposes. These basins occur either in granite or in tertiary sandstones and shales, and, as the fill is recent, the difficulty encountered by Mr. Smith in defining the basin seldom arises.

Both Messrs. Meinzer and Smith have commented on transpiration losses, and the latter has contributed the first really useful statement of basic principles which the writer has seen. It is to be hoped that further work will be done to confirm the principle of equal relative rates of transpiration, and that quantitative determination be made of the ratio between evaporation and transpiration. In all such experiments care should be taken to measure evaporation under some standard condition, for, as is well known, the results derived from pans in large bodies of water, in wet soil, in dry soil, and in air, differ widely, and in the same environment differ with atmospheric exposure.

In this connection the writer wishes to call attention to the fact that Mr. Smith's conclusion (page 1955†) that the combined loss by evaporation and transpiration from salt-grass sod is twice that from damp soil alone is not justified. The rate of water evaporation from the deep tank in the soil far exceeded that from soil during the winter, for the reason that the soil was kept cold by evaporation and cold air temperature, while the water was free to circulate and received warmth from the deeper soil. Experiments (which the writer carried on but did not publish) indicate that the annual evaporation from saturated base soil is about 93% of that from the water surface in the deep tank. The combined transpiration and soil evaporation for similar conditions was 115 per cent.

* *Proceedings*, Am. Soc. C. E., for April, 1914.

† *Proceedings*, Am. Soc. C. E., for August, 1914.

Mr. Lee.

With these general observations the writer will pass on to detailed comments on the discussion of the paper. Mr. Smith's paragraphs, grouped under the head of "Estimates of Safe Yield" (pages 1956-62*), will be taken up first.

1. The conditions on the slopes of the Catalina and Rincon Mountains, described by Mr. Smith, are not as similar to those of the east slope of the Sierra Nevada as would appear at first glance. Table 3 shows that the lowest elevation of the Sierra slopes is 6 500 ft., the average 7 500 to 8 000 ft., and the maximum elevation 12 000 ft. The similar slopes back of Tucson extend from an elevation of 3 000 to about 6 000 ft. The Sierra slopes do not support a luxuriant desert vegetation nor forest growth, the latter, especially, being spotted and sparse. The precipitation on the Sierra slopes is all in the form of snow, which melts and sinks into the porous mantle before the growing season commences. On the intermediate slopes of the Catalina Mountains, however, the precipitation occurs largely in severe summer storms, from which the run-off is rapid and the percolation is immediately available to vegetation. These differences are all such as to favor greater percolation on the Sierra slope. Furthermore, there are numerous springs at the base of the Sierra slopes, the flow of which, in many cases, can be measured in second-feet instead of cattle drinks. Hence, the writer still believes that the run-off coefficients used are not excessively high.

2. Charlies Butte is a low mound of lava near the margin of a shallow flow which advanced out over the valley-fill. The Butte is not the crest of a bed-rock projection, as Mr. Smith suggests, but is superimposed on the valley-fill. The spring is of the type described by the writer on page 844,† and unquestionably has its source in percolation from the outwashed slope.

3. The writer believes that Mr. Smith is justified in giving a value to transpiration from the luxuriant desert vegetation, the roots of which are within reach of the water-plane. The addition of this quantity would increase the computed ground-water discharge from the basin.

4. Relative to the matter of underflow from the basin past the Alabama Hills, the writer has given this matter considerable thought at various times, and cannot agree with Mr. Smith as to the probability of there being an appreciable loss at this point. For a distance of 6 miles from the "Point of the Hills" to Lone Pine Creek there are no lateral streams breaking through the hills. The whole cross-section was formerly covered by Owens Lake, and any material brought down by Hogback Creek would have been deposited on the lake shore at the "Point of the Hills", or, in case of low lake level, would have been

* *Proceedings*, Am. Soc. C. E., for August, 1914.

† *Proceedings*, Am. Soc. C. E., for April, 1914.

Mr. Lee. carried directly out on the lake bed. There would be no condition favoring or even rendering possible the deposition in the section of any but the finest materials, even on the side adjoining the steep slope of the Alabama Hills. Further, there is the elevated water-plane of the Lone Pine delta opposing such underflow, and the absence of any evaporating area which would naturally result from the back-water effect. Finally, there is the fact that the valley floor of the Independence Basin above the Alabama Hills is an old lake bed beneath which fine sands and clays predominate, that the porous gravels of the outwash slope surround this relatively non-porous core, that ground-water percolating laterally through the gravels meets a barrier at the old lake shore and is compelled to seek an outlet locally by spring flow and evaporation, and that, therefore, the conditions necessary for an active underflow down stream are entirely lacking throughout the whole of the valley floor.

The discussion of probable annual variations in safe yield is not of such great importance for the Independence Basin as in other basins. This is due to two reasons, the immense storage capacity of the saturated gravels and the non-porous core or heart of the valley. The former is relatively very large with respect to any judiciously developed draft, and the latter holds up the water-plane and prevents the escape of water from the basin by underflow. Thus, the fluctuation of the water-plane within a reasonable distance of the old lake shore would not be great, even in periods of extended drought. The effect of variations in ground-water supply in the Independence Basin would appear more as variation in spring flow and evaporation loss than as fluctuation in the water-plane.

Mr. Meinzer's long and intimate experience and scientific study of the problems involved in this paper make him one of the best qualified men in America to discuss it, and his contribution has added greatly to its value. The writer, however, feels that in general his discussion is from the point of view of the pure scientist who strives for absolute accuracy, rather than from that of the applied scientist or engineer whose aim is relative accuracy. There were involved in the solution of this particular problem not only questions of obtaining a proper internal balance in relative accuracy, but also that of giving the problem its proper place in the larger one of determining the safe yield from all sources available for a large municipal water-supply project. The quantity of ground-water available for development from the Independence Basin is about one-fifth of that available from all sources. The writer believes the results obtained, as set forth in this paper, are well within the limits of reasonable accuracy for the purpose desired. Considered in this light, he does not agree with Mr. Meinzer that the assumptions are "subject to large errors."

Mr. Lee.

Considering Mr. Meinzer's discussion in detail, the writer is indebted to him for drawing attention to the erroneous inclusion with ground-water discharge of 18 sec-ft. of irrigation water dissipated by evaporation and transpiration. As has been pointed out by both Messrs. Smith and Meinzer, however, there is an appreciable loss from rank desert vegetation bordering the shallow ground-water area. The inclusion of this with ground-water losses would increase the latter, and, therefore, the corrections in the final result would tend to offset one another.

Mr. Meinzer states that observations covering 1, 2, or 3 years do not give average evaporation conditions, comparing them with precipitation and stream-gauging data in this respect. The writer does not believe that this statement would have been made if evaporation records covering several years had been examined. The range of annual variation in evaporation is usually less than 4%, whereas precipitation and stream flow may have extreme variations of more than 100 per cent.

Mr. Meinzer suggests that, in order to apply the results of the Owens Valley evaporation experiments to other valleys, it would be necessary to have a large number of observation wells and keep them under observation for several years. The writer's experience in other valleys during the past 3 years has been that water-plane fluctuations in evaporation areas follow the same annual periodic law observed, and have about the same range as observed in Owens Valley. Hence the observation of a few judiciously chosen wells at a critical date, preferably late in September, should give dependable results.

The writer agrees heartily with Mr. Meinzer that further investigations of soil evaporation and transpiration under various conditions should be made. In connection with the rate of evaporation from bare alkaline lake beds or "palayas", certain experiments recently carried on under the writer's direction tend to confirm Mr. Meinzer's observations. The evaporation from two pans of water under exactly similar conditions was observed, one pan containing distilled water and the other a sample of highly alkaline lake water. The rate of evaporation from the denser water was less than that from the fresh water, and rapidly decreased to a very small value as the salts began crystallizing out, regardless of the fact that no permanent crust was allowed to form.

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SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

Discussion.*

By John H. Lewis, Assoc. M. Am. Soc. C. E.

John H. Lewis, Assoc. M. Am. Soc. C. E. (by letter).—Engineers are coming to realize the necessity for a more definite administration of streams by the public for the protection of existing and future acquired rights. They know what the law ought to be in order to accomplish the greatest good. Unfortunately, much confusion exists among lawyers as to just what the law really is, for any particular State, or for the Nation. Accordingly, a strong protest is made when any fundamental change is proposed. Little progress can be made in the reform of water laws, therefore, until such time as a large body of thinking citizens can come to some exact agreement as to just what legislation is needed, and then carry on a definite educational campaign to such end. Mr. Johnston's paper dealing with the fundamental principles of good water laws, therefore, should be a welcome addition to engineering literature. Mr. Lewis.

It is pointed out that recognition of the doctrine of public ownership of property in water is the necessary foundation for public control, and that beneficial use should be the basis for establishing and maintaining rights to the use of public waters.

These definite principles are rapidly being accepted as the only logical basis for an administrative code of water laws. The necessity for the adoption of such laws is becoming more apparent each day, as the larger and more expensive projects for the utilization of water are taken up. It would be folly to invest millions of dollars in an undertaking unless firm legal rights to the necessary water can be

* Continued from September, 1914, *Proceedings*.

Mr. Lewis. secured in advance of construction. Further progress in the utilization of our streams demands that definite laws be enacted.

The proper jurisdiction for the enforcement of such laws, however, is a matter of some uncertainty. Any discussion of fundamental principles, it seems to the writer, would be incomplete without some mention as to whether such laws should be administered by the County, the State, or the Nation, or perhaps by the drainage basin, irrespective of State lines. If we recognize the "public ownership of streams", then which of these political divisions shall act for the public in the administration of this public property?

Agreement as to this point seems to be fundamental. In the days of small projects and abundant water, county control seemed adequate. Later, State control was found to be necessary. With the taking up of vast irrigation, drainage, water-power, flood-control, and navigation projects, involving the expenditure of millions of dollars and the building of structures in two or more States, it would seem necessary, in such cases, that the water laws be administered by some authority superior to that of the individual State.

At the outset, when the total investment was small, little attention was paid to the question of legal title to water; nor was much consideration given to problems of administration. If subsequent appropriators caused injury, the wrongful diversion was restrained by injunction proceedings in the Courts. The cost of litigation to preserve prior rights became in time so excessive as to compel early appropriators to insist on the adoption of modern administrative codes for their protection. These laws proved a boon to capital, as definite grants could be secured which would ripen into vested rights when the water was put to use.

Litigation between States over interstate waters is increasing. Eventually, diversions in some States must be enjoined for the protection of water users in lower States. The loss of time and money in protecting States' rights on interstate streams must eventually reach appalling proportions. Capital will hestitate to invest for fear the new diversion when made will be enjoined. The same difficulties which have been encountered between appropriators within a State will be encountered between appropriators in different States. State laws, however perfect, can give little relief. Stagnation in development and the great loss in time and money resulting from such an archaic system of "government by injunction" will compel the adoption of some Federal administrative code of water laws in the future.

The administration of streams is a public necessity. Both State and nation have been derelict in their duty in this regard. Public rights have been shamefully trespassed upon in many cases for want of an administrative department with power to protect the public

interest. Beneficial use is the proper basis for such administrative laws, whether State or National. Mr. Lewis.

The unwise use of public property through lack of a definite administrative water policy for the greatest good to the greatest number occasionally results in great waste. This point may be illustrated by Snake River between Lewiston, Idaho, and Huntington, Ore., where the granting of a railway right of way may defeat the ultimate development of about 800 000 h.p. and the furnishing of water transportation to an empire greater than the combined areas of New York, New Hampshire, and Vermont.

Through lack of a public water department having full power to protect the public interests, we may wake up a few years hence and find that our public land department has granted land rights which will defeat public water rights of very great importance to the proper development of this region.

FIG. 1.

Between Lewiston and Huntington (Fig. 1) there is a fall of 1 300 ft. in Snake River, in a distance of 184 miles. The stream forms the boundary between Oregon and Idaho. At certain seasons it is navigable with great difficulty to Pittsburg Landing, 80 miles above Lewiston. This portion of the stream is doubtless under Federal control. Above this point, it is clearly non-navigable under present conditions, and is probably under the jurisdiction of the States with respect to water appropriations. One bank of this upper portion of the stream is in a National Forest, and the other bank for many miles is withdrawn from entry by the United States as being of great value for power purposes.

The river flows in a deep, rock-walled canyon, and for much of the distance is only from 300 to 400 ft. wide at the water surface. No foundation tests have been made, but conditions seem favorable for the construction of high dams and the ultimate development of about 800 000 h.p. Below the mouth of Salmon River, its low-water flow is approximately 10 000 sec-ft., and above that point 6 000 sec-ft.

On the profile, Fig. 2, there is sketched an ideal system of water-power dams arranged so as not to conflict with a 550-ft. tunnel project on Salmon River. This, with a 220-ft. dam and a 7- or 8-mile

Mr. Lewis.

tunnel, would develop 200 000 h.p., with the 4 000 sec-ft. of water available at low stage.

The lower dam, 175 ft. in height, would make 27 miles of slack water for navigation purposes to the foot of the next dam above. Ultimately, as the market for power increases, this stream, with small additional expense, could be opened to cheap barge transportation incidental to such water-power development. What this would mean to the 43 700 000 acres comprising the great Snake River Basin in Idaho and Oregon, is inconceivable. The United States Census shows 1 611 000 acres to have been irrigated in this district during 1909, and 3 907 000 acres are embraced in projects under way. The storage of water in Idaho for present use, together with diversions from the regular flow, has already injured to some extent diversion and pumping rights in Oregon. How the unrestrained use of water by Idaho in the future will affect Oregon cannot be predicted with the data at hand.

A water-power plant is now being constructed, presumably under State authority, by tunneling across a bend in the river just above Homestead, in Oregon. Will this plant be compelled to submit to Federal regulations at some future time, when the river is actually made navigable, or is declared so? How can Oregon protect its water supply against subsequent diversions in Idaho and Wyoming? Irrigation may be the highest use for water in the valley above, yet it is clear that water-power with incidental navigation is its only use in the canyon below. These are some of the uncertainties arising from the interstate character of the stream.

Although much of this power is supposed to be under the joint control of Oregon and Idaho, for all practical purposes it can be destroyed by the Federal departments controlling the public lands in this canyon, by the granting of a railroad right of way with the grade close to the water surface. Such a road has already been constructed from Huntington to Homestead, 58 miles, and right of way for the remainder of the distance has been granted. The construction of this road would add greatly to the cost of, if it did not entirely defeat, such a plan of water development, if found feasible, in the future. Snake River Valley now has transcontinental railway transportation, and the building of this cut-off on water grade would make but little, if any, difference in rates, compared with water rates. Whether it is feasible to locate such a proposed railroad several hundred feet above the water surface, so as not to conflict with the proposed plan of water-power development, can only be ascertained from a careful survey.

The rapidity with which the electric furnace is developing and opening up new possibilities for the utilization of vast quantities of electrical power, makes this project somewhat hopeful of materializing, from the standpoint of finding a market for the power. It is one which seems

Mr. Lewis.

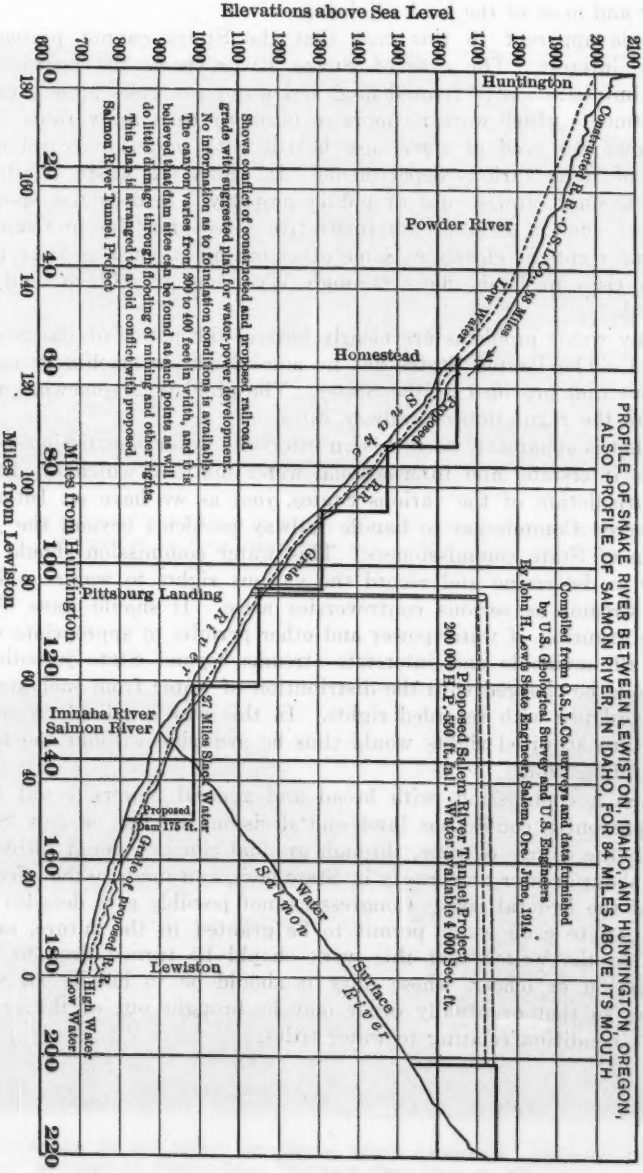


Fig. 2.

Mr. Lewis.

to be worthy of careful consideration by the public which owns the water and most of the land adjoining.

It is apparent in this case that the States cannot protect the public interest. The case of Snake River shows the confusion in the administration of related land and water resources among various departments which work no more in harmony than the various States. It shows the need of some new boards with power to correlate the work of these various departments. It shows the waste which may come through unwise use of public property. It does not show the need of such a Federal administrative board for the protection of existing rights as clearly as some other examples, such as Bear River, which rises in Utah, flows through Wyoming and Idaho, and ends in Utah.

Many water problems are clearly beyond the power of the State to control. The United States has no administrative machinery to supplement that provided by the States. The problem is somewhat analogous to the regulation of railway rates.

What is apparently needed is an interstate water commission to handle all interstate, and international water matters, which are beyond the jurisdiction of the various States, just as we have an Interstate Commerce Commission to handle railway problems beyond the jurisdiction of State commissioners. This water commission should have power to determine and record the various rights to water on interstate streams as serious controversies arise. It should have charge of the granting of water-power and other permits to appropriate water from all navigable and interstate streams beyond State jurisdiction. It should be charged with the distribution of water from such streams in accordance with recorded rights. In this way a reliable record of all future acquired rights would thus be available without the necessity of litigation.

Such a commission, with broad and general powers, could study the apparently conflicting laws and decisions of the various States and, in the course of time, through gradual changes, could ultimately bring about greater uniformity in State laws, and increase the effectiveness of the Federal laws. Congress cannot possibly give detailed consideration to each water permit to be granted in the future, and it seems to the writer that this work should be turned over to some commission or officer, whose duty it should be to handle all water matters so that eventually order may be brought out of the present chaotic condition relating to water titles.

FIG. 3.—PERILS OF NAVIGATION IN SNAKE RIVER CANYON. THE GASOLINE BOAT *Prospector*, IN FOUR DAYS, REACHED A POINT MORE THAN 700 FEET VERTICALLY ABOVE LEWISTON.

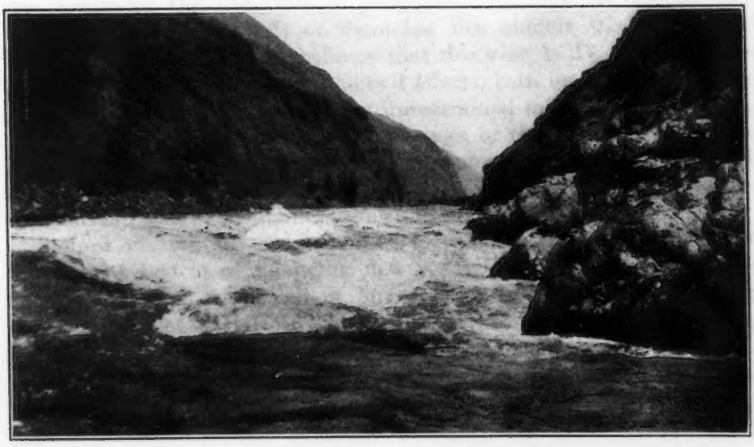
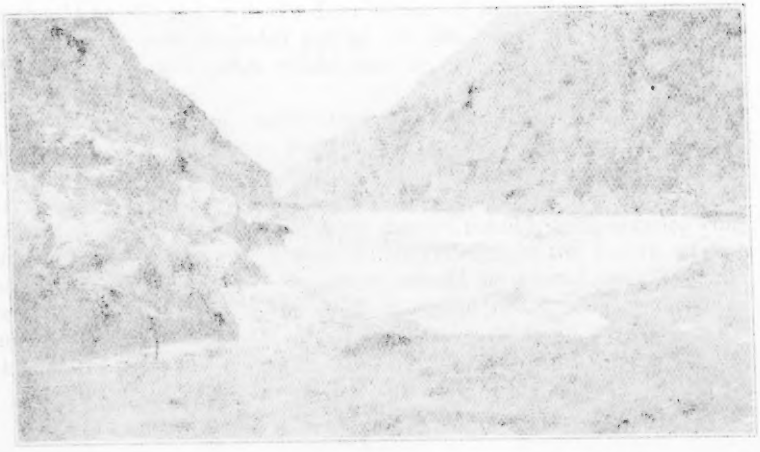


FIG. 4.—SNAKE RIVER, ABOUT 100 MILES ABOVE LEWISTON. HIGHEST POINT REACHED BY POWER BOAT, WHERE 9 FEET FALL IN 100 FEET PREVENTED FARTHER PROGRESS.



AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By J. P. Hallihan, M. Am. Soc. C. E.

Mr. Hallihan.

J. P. Hallihan, M. Am. Soc. C. E. (by letter).—Throughout the discussion on the report of the Committee, there does not appear to be any mention of the shrinkage in the value of railroad securities that is now taking place and the apparent necessity of devising some method of financing for the future that will tend to stabilize those values; a method that will not curtail enterprise, while reducing to the minimum the inflation of securities. It may be argued that the discussion of methods of financing lies outside the province of this Society, but the writer believes that this view will be taken by few engineers who have witnessed the evil effects, both on the property and on the security values, that follow unrestrained manipulation of securities and capitalization of surplus earnings, or the further and perhaps more disastrous results in retarded development of resources that will follow a regulation of roads and security issues, that eliminates the speculative profit without increasing the safety of the investment.

Before offering any suggestion as to changes, it is well to consider the present system of financing new roads, a system which has been applied to all or nearly all of our present railroads. Theoretically, the securities of a railroad should produce cash in amount equal to the face value less the usual bankers' and brokers' commissions, and this situation may at times be approached in the case of extensions by roads long established in profitable territory, maintained to a high standard, and with surplus earnings sufficient to guarantee interest on bonds and to assure the payment of dividends on stock.

* Continued from August, 1914, *Proceedings*.

Mr. Halliihan.

In the case of a new road these guaranties cannot be furnished. In support of the securities there exist only partly developed resources —potentialities—which, combined with personal effort and capital, may be moulded into a state of productiveness sufficient to support a railroad.

Where is the required capital to be obtained? Those having the most faith in the enterprise and whose property will be most benefited by it, have already done their best to make it possible by carrying on the initial development of resources and, in this effort have expended all their capital. Those living in more prosperous communities, supplied with transportation, and still sensible of the benefits to be derived from the use of transportation facilities, are also mindful of the hazards attending the early days of that form of investment, and confine their purchase of securities to those of going concerns. The enterprise, therefore, is forced into the general money market with a security known as a construction bond carrying a low rate of interest and having behind it only a charter, some surveys and estimates, and the good-will and confidence in themselves—of their ability to fill the measure of their promises—of the communities desiring the facility.

In competition with high-class securities, supported by visible assets, the construction bond is hopelessly handicapped, and the enterprise is forced to seek the help of some financial entity strong enough to advance the necessary capital against the bonds and to assume as well the responsibility of creating the assets on which the bonds are to rest.

What is the value of this service, which in effect is the vitalization of the enterprise?

Naturally, any banking house furnishing the capital for a new project must be put into a position to control the expenditures, as it assumes a certain responsibility toward the individual purchaser of the securities. This requires actual stock control, so that at the inception of the enterprise at least 51% of the stock must be issued to the bankers, or trusteed in some manner so that they obtain voting control.

It has been the practice in the United States to deliver the bonds to the bankers at a discount sufficiently below the probable selling price to the public to allow for bankers' and brokers' commissions, and to turn over to them as well the entire issue of capital stock less such portion as promoters may be permitted to retain for their services in bringing the project up to the point where it will stand financial analysis.

The banker does not retain all this stock, for, in disposing of the bonds to his clientele, he, in turn, is forced to offer stock as a bonus in order to attract investors from the more conservative channels of

Mr. Halliban.

investment, for even at a liberal discount and supported by the endorsement of a reliable banking house, a construction bond is always considered a speculation and the purchaser is entitled to a share in the speculative profits of the enterprise, in addition to the interest earned on the bonds.

Inasmuch as no actual money has been paid by any one receiving this stock, it appears that something has been given for nothing, but an analysis of the comparative values shows that, if value is measured by the market quotation and demand, neither stock nor bonds have more than a nominal value in the market, nor can they have until the enterprise is carried through to a successful termination. In the meantime, the bankers, willingly or otherwise, must share with the promoters the responsibility for the project, and must be prepared to support it through the vicissitudes of the construction period and of the early years of operation, or the development period.

The stock, therefore, must be considered as a promise to pay at some future date for the brains, personal effort, and credit (as fitly stated by Gardner S. Williams,* M. Am. Soc. C. E.) utilized in creating and administering the property. Both in time and amount, this payment is directly dependent on the success attending the use of the faculties mentioned. If unsuccessful, the consideration is of no value. If successful, those who take the risk are entitled to a reasonable proportion of the fruits of that success.

When the enterprise emerges on a self-sustaining basis and earns a surplus, over fixed charges, sufficient to enable a dividend to be paid on the stock, a market is created and the stock now becomes a speculative investment, gradually passing from the original financiers into the hands of the general public.

Up to this point, useful work has been performed by the promoters and bankers, new wealth has been created, and the risk assumed by the capital has been remunerated to the satisfaction of every one concerned, including the actual users of the service. Thus far, the public, regarded as the user, has suffered no injury. It did not participate in the original investment, assumed none of the risk, and helped only passively to create the value. A high price has been demanded for the capital used, but the payment of this price was justified by the narrowness of the market for the securities offered and the pressing need of the facility. At a less price, or without the attraction of a speculative profit, the capital could not have been obtained unless the risk was reduced by increasing the local equity, or by some form of interest guaranty, both usually impracticable.

Theoretically, the public, having now purchased a majority of the stock, is the owner of the road and is responsible for its administration and for the disposition of its earnings. Actually, the road

* *Proceedings*, Am. Soc. C. E., May, 1914, p. 1407.

Mr. Hallihan.

is effectively held by the original financial group, now very greatly in the minority as to stock ownership, having taken their speculative profit; but a minority highly organized, which has named the directors, installed the management, and decides the financial policy. The buying and selling of stock on the exchange rarely disturbs this control, but if it is disturbed, it merely passes to another similarly organized minority.

Certain abuses of this power, in the way of questionable financial deals conducted by inside syndicates, distribution of unearned dividends to assist the manipulation of securities, and a general exploitation of properties with disregard of the interests of the stockholders and the users, have been followed by regulatory acts, which, at times, may have merited the criticism of being too drastic, since it has been impossible to differentiate between well managed and badly managed properties.

The work done by the Interstate Commerce Commission, however, has been along constructive lines. It has benefited the position of the user of transportation by relieving him of unfair discrimination and kindred evils; it has constituted the Commission the balance of power between the large and the small stockholder, or between the organized minority and the unorganized majority, and in this respect has materially improved the security market, because it has created in the mind of the small investor a feeling that his investment is being looked after by a disinterested arbiter, and has thereby widened the market for railroad securities—a distinct benefit to the bankers. The valuation of public utilities (though there may be some difference of opinion as to such valuation being the proper method for determining rates) is nevertheless bound to effect a wholesome change in the disposition of the earnings of railway properties. All this tends toward establishing security of investment, and is, therefore, constructive work.

The restrictions surrounding bond and stock issues of new transportation projects, however, have practically eliminated the speculative profit that was offered to capital when our present railroad systems were built, with the result that capital hitherto utilized in developing transportation facilities has been directed to other and more attractive forms of investment or has found use abroad.

It is the writer's belief that this is a condition of sufficient economic importance to warrant the consideration of the Society in connection with the valuation of public utilities, as there is still a large area of our country unprovided with transportation facilities and rapidly developing to a point where the inhabitants will have a right to demand that such facilities be supplied. Before money can be obtained for such purposes, the differences of opinion now existing between the various regulatory bodies and the bankers must be harmonized

Mr. Hallihan.

and a financial basis arranged, which will at once protect the user of transportation and provide adequate return for the risk assumed by the financier and investor at the various stages of the enterprise. The questions to be considered appear to the writer to be as follows:

1st.—What, if any, investigation should be made by the Interstate Commerce Commission of the necessity for a transportation facility prior to issuance of a charter by the State?

2d.—What limitation per mile should be placed on the bond issue for single-track roads in:

a. Territory yielding a light traffic?
b. Territory yielding a heavy traffic?

3d.—What discount should be allowed on the bonds?

4th.—What proportion should the share capital bear to the authorized bond issue?

5th.—What proportion of the share capital should be allotted to the bankers and promoters in payment for their services and credit—to insure control and as speculative profit? At what discount should the remainder be offered to the public?

6th.—If speculative profit is not allowed, should any guaranty be given by the Government in any form to reduce the risk to an investment basis, and what reversionary rights should the Government receive in exchange?

7th.—What provision should be made for amortization of bonds?

In asking this latter question, the writer suggests that a difference of opinion may reasonably exist as to the definition of the word "depreciation." If depreciation is taken to mean the difference between the present value of an article in use and the new value of the same article, it is apparent that a fund set aside for the purpose of maintaining that article in a condition of 100% efficiency cannot be used to provide for replacing it with a tool of double the efficiency and costing twice as much. The necessity of replacing an old article with a superior one is due to the fact that the old article has become obsolete. A 60-lb. rail, on untreated ties, on an unballasted roadbed, is not replaced with a 60-lb. rail, but with a 90-lb. rail, on treated ties, on a well-ballasted roadbed. The mere maintenance, therefore, of the original rail in a state permitting it to perform the service which may be performed by a 60-lb. rail may call on the earnings for just the amount necessary to maintain that condition and to replace it when worn out, and no more. To supply the additional and further amount that is required to bring the transportation tool up to a present-day efficiency, new capital must be provided, whether out of earnings or otherwise.

When the replacement is made a value will be wiped out, which is the difference between the scrap value of the old rail and its

Mr. Hallihan.

efficiency value when laid on the roadbed and performing its work. This value has been used in the performance of its work, and at the time when the rail is taken up, the same value should have disappeared from the original loan. That is, the bonds should have been reduced in amount covering this loss; or, in other words, the borrowing capacity of the road must be kept up to provide the extra capital required for obsolescence.

The discussion apparently indicates that both these quantities are assumed to be included in the one account—depreciation—but it would seem that for the sake of clearness in accounting, it would be desirable to create a separate fund for amortization of bonds.

8th.—What is a fair return to the stockholder?

9th.—In the case of roads operating under competitive rates, but still earning a surplus over a fair return, should the surplus be permitted to accumulate as a surplus for ultimate division among the stockholders as extra dividends, or be applied to an interest reserve fund for extensions and betterments?

The writer suggests discussion of these questions in the belief that the Society, from the experience and training of its members, is well qualified to deliver an impartial opinion and to formulate standards of practice that may serve to establish railroad investments on a more attractive as well as on a sounder financial basis.

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DISCUSSION OF CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS.*

By Jules E. Hanique, Assoc. M. Am. Soc. C. E.

Mr. Hanique.

Jules E. Hanique, Assoc. M. Am. Soc. C. E. (by letter).—This problem is of vital importance, as its solution will affect the majority of engineers of all classes. The revised and enlarged tables which the Committee is now preparing should yield very important and needed information.

The writer has read all the discussions to date, and wishes to concur in the opinion expressed by Messrs. Phelps, McCullough and Fuller, as to the necessity for a "clearing bureau," maintained by the Society, for the benefit of all the members.

Neglecting the higher positions, held by those whom Mr. Fuller and Mr. Henoch call the successful engineers living in an environment of "big things", the writer would like to make a few remarks on the condition of the "submerged ten", and show some reasons for their "unethical conduct in seeking work at a wage below the standard."

It is important to make a difference between an engineering position and a job of engineering; inasmuch as a "position" means work at a fixed salary, usually on the staff, and the incumbent enjoys many advantages. After working hours he is free to associate with whom he pleases; he can read up on any pet subject; he has access usually to some good library; but the important point is that his position, being of a permanent nature, he can arrange his budget of expenses and regulate his income almost to a certainty for a long period of time. He can invest his savings to advantage, and, being

* Continued from May, 1914, *Proceedings*.

Mr. Hanique.

on the ground, can also watch them more closely. His leisure can be arranged so as to give him the maximum benefit, through his reading, studies, and principally through the careful selection of his associates, which can be made to yield large returns.

Men of the other class, they who follow "job engineering," have an entirely different "row to hoe." The experience of many such has been summed up by Mr. Fuller. Twenty-four hours' notice is not unusual, and, in some cases known to the writer, less time has been given. Is it any wonder that the men thus dismissed are badly handicapped? If the man has a family, it is still worse; he must move at once to some large center from which to start a new campaign; he must go around again and pick up his old acquaintances; he must, patiently and cheerfully, go from one man to another, up and down stairs and elevators, in and out of offices, past clerks and stenographers, and brave the withering looks of office boys and prospective employers; but he must remain neat, presentable, and, above all else, cheerful.

He knows that his savings are running low, and that prospects are not very good, but still he must keep on looking. The situation becomes critical. Under the circumstances, what is he to do? Work at anything, until something turns up; and who is there to blame him? It may be unethical, but it is eminently necessary and certainly practical.

It is very well in times of plenty to talk of standard wages, but it is to be borne in mind that, when work is plentiful, no engineer, nor any one else for that matter, will work for less than the market value of his class of work.

The salaries for engineers is one of the mysteries of the age; some attempt has been made at fixing salaries for different classes of work, but on what basis? A muck foreman's wage is determined by the amount of work he can get out of his gang; a general foreman will receive a salary greater than that of the engineer on the job; and the engineer is supposed to know the work of all the foremen, in addition to his own. Why should this be? What becomes of the "prep." school work? What of the four years of collegiate work? What is the incentive to the young man in the face of these conditions?

The Society, in so far as the welfare of its members is concerned, fails just when it is most needed, as a not too liberal interpretation of Section 3, Article I, of the Constitution will show.

The Society has many local branches scattered about the United States, and it seems to the writer that through them it would be feasible and inexpensive to maintain a register of positions open in the particular district in which such local branch is situated. All unemployed members could have access to this register on payment of a fee large enough to reimburse the local association for any expenses it may have incurred.

Mr. Hanique.

The writer assumes that members of the Society in good standing would be given the preference, because the fact of their being a member should be *prima facie* evidence of ability, or at least of experience. If many are incompetent, as has been suggested, the fault lies in the admission requirements, and these should be changed.

The writer sees nothing in these suggestions tending to lower the dignity of the Society or of the Profession, and earnestly commends the foregoing to the attention of all members, believing that the subject is too big and too vital to be classed as mere academic discussion and tabled.

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The writer sees nothing in these suggestions tending to lower the dignity of the Society or of the Profession, and certainly commends the foresight to the attention of all members, believing that the subject is too big and too vital to be classed as mere academic discussion and tabled.

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THE CONSTANT-ANGLE ARCH DAM.

Discussion.*

By Messrs. D. C. Henny and Edward Wegmann.

Mr. Henny.

D. C. Henny, M. Am. Soc. C. E. (by letter).—The economical design of arch dams in canyons of considerable width at the bottom and with practically vertical sides, calls for a constant angle and constant radius. The author deserves full credit for pointing out that in canyons which are more distinctly V-shaped the design requires a variable radius and as far as possible a constant angle in order to obtain maximum economy.

It does not need much mathematical analysis to be convinced of the general correctness of his reasoning, and the writer is in part responsible for the adoption of a similar design for a dam of this type, now under construction.†

With the more detailed reasoning in the author's analysis of interior stresses throughout the body of masonry, however, the writer cannot fully agree, largely because he does not believe that the facts correspond entirely with the assumption on which the analysis is based.

It has been assumed by the author, as is customary in studies of this kind, that the weight of the dam is transferred vertically downward to the foundation, and that the water pressures against the structure are transferred horizontally to the abutments. A division of the loading is thus arrived at between the vertical cantilever and the horizontal arch, and Poisson's ratio is further applied in an attempt to approximate more closely actual stresses and deformations.

* This discussion (of the paper by Lars R. Jorgensen, M. Am. Soc. C. E., published in May, 1914, *Proceedings*, and presented at the meeting of September 16th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Clear Creek Dam, Washington.

Mr. Henny.

This method of reasoning may be logical in the case of a dam almost as long at the bottom as at the top. The principal subject of the paper, however, is believed to refer especially to dams with V-shaped rather than U-shaped longitudinal sections, and to such dams this reasoning is not believed to be logically applicable.

In the matter of the distribution of foundation pressures due to gravitation alone, a difficult problem is presented in which the shape of the canyon and the method of construction become important factors.

If a dam is built as a monolith and brought up horizontally, with the layers well connected or interlocked by large rocks, and if the sides of the canyon are steep, the dam becomes to a large extent a wedge, and the load will be partly transferred from the middle to the sides of the canyon through combined arch and beam effect. Thus, the loading at the bottom of the canyon will become greatly reduced.

If, on the other hand, the dam is built with vertical contraction joints, the horizontal load transference will become reduced, although friction along such joints, if under heavy pressure through lateral expansion due to vertical loading and through horizontal water loading, will still leave considerable beam and arch effect.

If the dam is built up in vertical slices with large vertical joints between, and these joints are finally filled, under the most favorable conditions of temperature, not only will lateral pressures from vertical loading be almost completely nullified, but entirely different conditions as to initial stresses will exist.

The effect of the shape of the canyon and the difference in methods of construction will be equally important as to transference of horizontal water loading.

If Fig. 18 represents the developed longitudinal section of an arch dam, it is evident that in a dam of that type, the shortest distance of a point *a*, in the mid-vertical cross-section, to the abutment does not lie in a horizontal, but in an inclined direction, approximately normal to the slope of the canyon. A down-stream movement of the point *a*, such as may result from water pressure, therefore, will produce relatively more deformation and compression in the direction *ab*, than in the direction *ac*.

In transferring water loading from the middle to the ends of a dam, the stresses or lines of pressure, therefore, will be deflected through weight and possibly tension into curved lines, and thereby follow shorter than horizontal lines to points of support, as indicated by the dotted lines on Fig. 19.

If *ede* represents such lines of pressures, then the arch which mainly transfers the load, will have its crown at elevation *d* and its springing lines at the lower elevation, *ee*, and if developed will ap-

proximate an elliptical shape, with largest radius of curvature and with least thickness at the crown. Mr. Henny.

If these pressure lines are representative of stress distribution, it is evident that there will be an increase in intensity of pressure near the top of the middle section and a corresponding decrease at the bottom.

If the dam is not built as a monolith but with contraction joints, there will be a lesser tendency to deflection from the horizontal; but, through friction in connection with initial and loading stresses, much of this tendency will remain, and it is quite probable that even then the pressures in the lower part of the dam, both up stream and down stream, may be profoundly affected by load transference.

In any study of division of load between arch and cantilever, moreover, it should be remembered that any inclined plane from the canyon walls to the top of the dam may develop a section which, considered as a cantilever, may, because it is shorter, be stronger and, therefore, have greater effect in opposing true arch action than the vertical cantilever on which the author's reasoning is based.

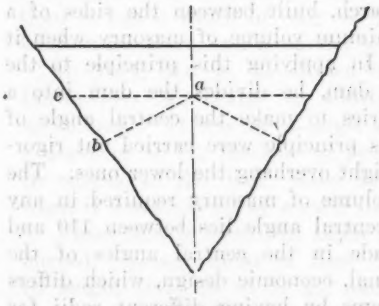

FIG. 18.

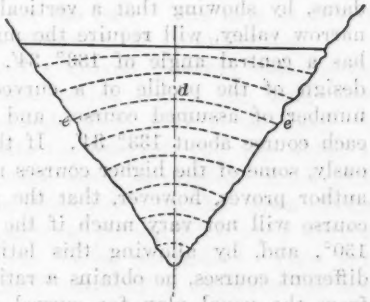

FIG. 19.

To summarize, the writer holds that if an analysis of interior stresses in a dam placed in a V-shaped canyon is to yield satisfactory results, it is essential that the section of the canyon along the center line of the dam should be made a factor in the problem.

This discussion is submitted with a full appreciation of the principal point brought out in the author's valuable paper.

EDWARD WEGMANN, M. AM. SOC. C. E.—The speaker will review briefly the opinions about arch action in curved dams, that have been advanced by different authorities. M. Delocre, a French engineer, investigated this subject in 1866, and reached the conclusion that a curved dam would act as a dam, providing its thickness at the base did not exceed one-third the radius of its up-stream face.* Another Mr. Wegmann.

* "Mémoire sur la forme du profil à adopter pour les grands barrages en maçonnerie des reservoires," par M. Delocre. *Annales des Ponts et Chaussées*, 1866, 2e Semester, p. 212.

Mr. Wegmann.

French engineer, M. Pelletreau, placed the limit of the thickness of the base, of a dam which will permit arch action, at one-half the radius of the up-stream face.* Professor Rankine, M. Krantz, and some other eminent engineers thought there were so many uncertainties involved in the assumptions made in the mathematics of arch action in dams, that they recommended that a masonry dam should always be designed so as to be able to resist, by its weight alone, the pressure of the water, and that, in the case of a narrow valley, the plan of the dam should be curved, as an additional safeguard, to utilize arch action.

During the preparation of the plans for the Lake Cheesman Dam, in Colorado, Silas H. Woodard, M. Am. Soc. C. E., made a very interesting mathematical investigation of arch action in curved dams.† This subject was carefully studied, also, taking into account stresses caused by changes of temperature, by George M. Wisner and Edgar T. Wheeler, Members, Am. Soc. C. E., in connection with the plans for the Pathfinder Dam.‡

Mr. Jorgensen has taken a step in advance in the study of curved dams, by showing that a vertical arch, built between the sides of a narrow valley, will require the minimum volume of masonry when it has a central angle of 133° 34′. In applying this principle to the design of the profile of a curved dam, he divides the dam into a number of assumed courses, and tries to make the central angle of each course about 133° 34′. If this principle were carried out rigorously, some of the higher courses might overhang the lower ones. The author proves, however, that the volume of masonry required in any course will not vary much if the central angle lies between 110 and 150°, and, by allowing this latitude in the central angles of the different courses, he obtains a rational, economic design, which differs from the usual plan for curved dams by having different radii for the up-stream face at different levels. The method is novel, and is theoretically correct.

The author states, on page 1287,§ that:

"Arch stress is distributed nearly evenly over the whole section, whereas the stress due to gravity action is very poorly distributed, being a maximum along the down-stream face, and diminishing to approximately zero at the up-stream face."

The speaker cannot fully accept this statement. Owing to the elasticity of the masonry, a curved dam will yield slightly when subjected to water pressure. This motion will be greatest at the center of

* "Mémoire sur les murs qui supportent une poussee d'eau"; par M. Pelletreau. *Annales des Ponts et Chaussées*, 1876, Octobre; 1877, Août et Novembre.

† "Analysis of Stresses in Lake Cheesman Dam"; by Silas H. Woodard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII, p. 108.

‡ *Engineering News*, August 10th, 1905, p. 141.

§ *Proceedings*, Am. Soc. C. E., for May, 1914.

Mr. Wegmann.

the valley and practically zero at the sides which form the abutments of the arch. It follows, therefore, that although we can calculate the compression in the arch by the simple, well-known formula given on page 1288,* this stress may not be uniformly distributed, as the slight yielding of the arch will cause the line of pressure to shift nearer to the up-stream face. In the present state of knowledge, it would be difficult to say whether the distribution of stress in the dam, considered as an arch, varies any less than that which is usually assumed to be caused in the masonry by the action of gravity.

Turning from theory to practical examples, we find that the plans of the old Spanish masonry dams, which were built more than 300 years ago, are curved or polygonal in plan, being convex up stream. About 1843, before any correct theory had been advanced for the design of masonry dams, M. Zola built a curved masonry dam to form a reservoir for the water supply of the City of Aix, in the southern part of France. This dam, known as the Zola Dam, is so thin that it cannot resist the water pressure by its weight. Its stability is solely due to arch action. For a height of 120 ft. above the foundation, it has a width of only 42 ft. at the base. The dam is 205 ft. long on the crest, and is curved to a radius of 158 ft. at the top.

In 1884, Mr. F. E. Brown built a curved masonry dam in the Bernardino Mountains in California, which equals the Zola Dam in boldness. It was 64 ft. high and only 20 ft. wide at the base. At a depth of 48 ft., the line of pressure for reservoir full was about 13 ft. outside of the dam. This structure stood successfully until 1910, when it was replaced by a safer dam, about 200 ft. farther down stream.

From 1896 to 1906, Mr. Leslie Augustus Burton Wade built thirteen curved, concrete dams in New South Wales, Australia.† They vary in height from 25 to 87 ft., and their masonry is subjected by arch action to maximum pressures of from 20 to 25 tons per sq. ft. The highest of these dams has a width of base of 24 ft. for a height of 87 ft.

At the end of his paper the author mentions three dams which have been built successfully on the constant-angle arch principle. The profiles of these structures are rather peculiar, but this is due to the author's attempt to get a better distribution of stresses and a smaller volume of masonry than are to be found in the ordinary curved dam.

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* *Proceedings*, Am. Soc. C. E., for May, 1914.

† *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 1.

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SUBAQUEOUS HIGHWAY TUNNELS.

Discussion.*

By J. V. Davies, M. Am. Soc. C. E.

Mr. Davies.

J. V. Davies, M. Am. Soc. C. E.—In working out the study of any individual case that is presented, the thing that particularly impresses the speaker more than anything else, is the great scope for the display of engineering ability in the various special lines of the Profession. In the last few years, in connection with the Interstate Bridge and Tunnel Commission, the speaker has discussed at public hearings the relative advantages of bridges or tunnels for vehicular traffic across the Hudson River to such an extent that the subject has become almost tiresome.

There are a great many marked advantages in certain particular cases in the use of tunnels for subaqueous highway purposes over the use and adaptation of the bridge: they are better protected; they present less external obstruction; there is less cost of maintenance, which is a very large item; and per track or per roadway, they will probably cost materially less. Another obvious advantage is that a bridge has to be constructed in the first instance for the full permanent use and purpose for which it is required; whereas, in the case of tunnels, single roadways can be constructed and added to as the volume of business extends, and such additional tunnels can be located and distributed as traffic requires.

It is certain that there will be a growing use of tunnels for highway purposes. The speaker, while in London recently, was rather interested in the fact that the Rotherhithe and Blackwall Tunnels

* This discussion (of the paper by George Duncan Snyder, M. Am. Soc. C. E., published in May, 1914, *Proceedings*, and presented at the meeting of September 16th, 1914) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Davies.

had been closed entirely to traffic, due to the fear of war conditions, involving the use of bombs or explosives which might cause serious injury to those structures. Reference has been made to a pertinent question, arising from the conditions developed during the procedure of war in Europe, as to the destruction of bridges across rivers by the contending armies, and it is really a matter of thought, whether a tunnel in such cases—with the ability to flood it and render it temporarily useless without permanent injury—has not a very great advantage over a bridge for ordinary highway purposes.

On the question of the transporter bridge the speaker has studied very closely three of the largest of such bridges—those at Runcorn and Newport, in England, and the one at Rouen, France. At all these bridges there is considerable rise and fall of the tide. At Newport, particularly, the average rise and fall of the tide in the River Usk, where the bridge is located, is some 35 ft. When the water is at high level the river is a very fine and noble stream. At low water there is a mud valley with a small creek at the bottom of it. A ferry service is impossible. An ordinary highway bridge would be difficult and expensive to build, and even with draw-spans would be a serious interference to navigation. A tunnel, on the other hand, on account of the banks and river valley, would require such approaches as would render it undesirable for such purpose; and where the traffic is very small, as it is in this case, and as it is also in Rouen and Runcorn, there is no warrant for constructing any very large and expensive draw-bridge, or, on the other hand, any expensive tunnel, for such conditions; whereas, the transporter bridge, giving a fixed end landing and reasonably frequent service, at low initial cost, serves the purpose of transportation admirably. This is an illustration of the fact that each individual case must be considered on its own merits.

The greatest point of advantage in the use of a subaqueous tunnel, over any other method of conveyance for highway uses, is its flexibility of location. The Rotherhithe Tunnel does not go squarely across the river; even the approaches are at varying angles; and the location and design of the tunnel were evolved to connect two great populous districts which had business intercourse and business interests on each side of the Thames, where that intercourse was urgently necessary. The tunnel, therefore, was constructed on a zigzag alignment. Crossing the Thames on a diagonal line, it allowed communication to be had between districts that required that communication, and, to the speaker's mind, more than anything else, that flexibility of location for highway purposes is one of the most important features in the consideration of this subject.

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EXTERNAL CORROSION OF CAST-IRON PIPE.

Discussion.*

By C. P. Bowie, Assoc. M. Am. Soc. C. E.

Mr. Bowie.

C. P. Bowie, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest. Although the author has dealt primarily with cast-iron pipe, some of the methods for the prevention of external corrosion would apply equally to steel pipes. In California a number of companies are operating 8 and 10-in. oil pipe lines which pass through many miles of semi-arid lands where the soils are strongly impregnated with alkali. The total salts in these soils may be said to range from 0.05 to 5%, the average composition of the alkali salts being about as follows:

Potassium sulphate	11.28%
Sodium sulphate	19.48%
Magnesium sulphate	2.89%
Sodium chloride	23.03%
Sodium carbonate	40.96%
Sodium phosphate	2.08%
Sodium nitrate	0.28%
	100.00%

Soil taken from the immediate vicinity of a pipe which had been in the ground for nearly 4 years and was badly pitted showed the following composition:

Sodium carbonate	0.02%
Sodium chloride	0.05%
Sodium sulphate	1.31%
Magnesium sulphate	0.14%

* This discussion (of the paper by Marshall R. Pugh, M. Am. Soc. C. E., published in August, 1914, *Proceedings*, and presented at the meeting of October 7th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Bowie.

The pipe lines are of wrought steel, about $\frac{5}{16}$ in. in thickness, tested in the factory to withstand a working pressure of 1 200 lb. per sq. in., although in practice they are seldom subjected to pressures exceeding 800 lb. per sq. in., and general operating conditions call for an average working pressure, at the pumps, of about 600 lb. per sq. in.

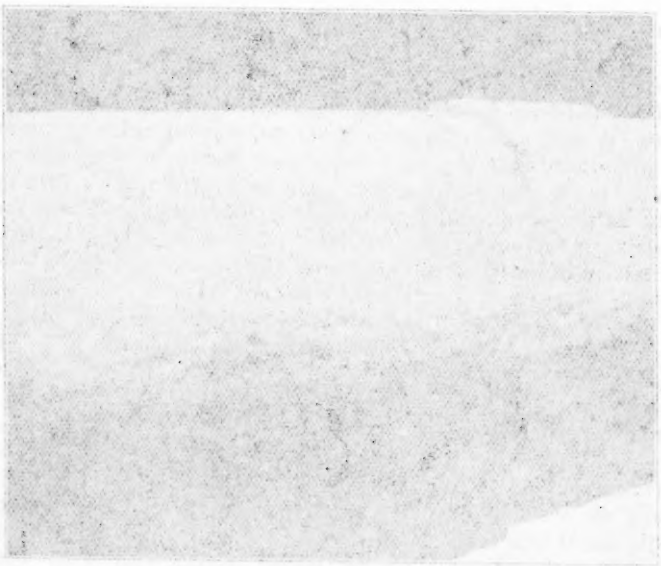
Instances are on record where such pipe, although originally protected with three coats of asphaltic paint, has corroded so badly in 3 years that the pit holes extended entirely through the metal. It is interesting to note that these pit holes often occur in portions of the pipe where the surrounding area shows little if any sign of corrosion. They are usually from $\frac{1}{2}$ to $\frac{3}{4}$ in. in diameter at the top, and often have the appearance of having been gouged out of the metal. Fig. 25 shows a number of such pittings in an 8-in. pipe which had been treated with two coats of asphaltic paint when laid, and had been in the ground a little more than 4 years. Fig. 26 shows a cast-iron saddle used for repairing pit holes which have developed into leaks.

Although in some cases adjoining property owners would have no objection if the pipe lines were carried on trestles above the surface of the ground, as Mr. Pugh states was done with the 48-in. pipe at Atlantic City, this method causes serious difficulty when it is considered that oil is pumped through these lines at temperatures (at the pumps) as high as 160° Fahr., and that insulation against the loss of this heat, such as is afforded by an earth covering of 2 ft. or more, is often essential to their successful operation, especially during the winter. Where the lines are placed on trestles of any extended length, the necessary insulation must be supplied, and this is expensive, even under the most favorable conditions. It is worthy of mention in passing that in places where a continuous length of a mile or more of pipe has been carried on trestles, across low swampy lands abounding in bulrush or tule growth, an almost ideal insulation has been effected by enclosing the line in a boxing of 2-in. plank, the space between the inner walls of the box and the pipe being stuffed with the dried tule.

In reference to the use of concrete as a protection: this method was tried a number of years ago by one of the large companies, but without success. The expansion and contraction of the pipe, caused by the high temperature of the oil and the relatively low temperature attained during the occasional necessary shut-downs, which allowed the pipe to cool to the temperature of the ground, soon cracked the concrete covering so badly that it was rendered practically useless. Moreover, collar leaks, which are of frequent occurrence in such lines, were not only difficult to repair, but as they necessitated the cutting away of the old concrete at such points, presented an even more serious defect, in that it was found practically impossible to

FIG. 25.—PIT HOLES IN 8-INCH STEEL PIPE, DEVELOPED IN A LITTLE MORE THAN FOUR YEARS.

FIG. 26.—CAST-IRON SADDLE USED TO STOP LEAKS DUE TO PIT HOLES.



put on patches of concrete without having cracks develop between the old and new material. Mr. Bowie.

A successful covering for such lines should be so elastic that it would expand and contract without rupture, in conformity with the pipe; it should afford insulation against loss of heat, and at the same time be impervious to the passage of water. Working along these lines, a number of coverings have been brought forward. Various kinds of asphaltum paint have been used, but, thus far, at least, without any marked degree of success. Crude oil has also been applied, by covering the pipe, after it has been lowered into the trench, to a depth of 6 in. or more with the oil, and then immediately filling in the trench with earth. This method is only partly successful, because, if the soil is at all sandy, the oil soaks into it and in time almost entirely disappears from the vicinity of the pipe, leaving the metal quite as badly exposed to the alkali as before the application. Quick-setting bituminous enamels, which can be put on in a layer from $\frac{1}{8}$ to $\frac{1}{4}$ in. thick, bid fair to give results. However, up to the present, these enamels have one serious drawback, in that they cannot be applied successfully to a moist pipe or to one having a temperature lower than 60° Fahr., because the enamel, although it is put on at a temperature of approximately 200° Fahr., is chilled so rapidly that it hardens before it can be spread. The action of the mop with which it is applied is merely to roll it into a myriad of balls or globules, which do not stick to the pipe but fall to the ground. This not only causes a great waste of material, but is apt to leave many small portions of the pipe, especially on the under side, wholly unprotected, even though several coats of enamel are put on. This difficulty, however, can be largely overcome by applying the enamel in warm dry weather only, or to pipes through which warm oil is being pumped. There are no lines in the State which have been covered by this process and allowed to remain in the ground for any great length of time. One line, passing through slightly alkaline territory, which was covered in this manner about 4 years ago, was found to be in excellent condition on recent examination.

The method of covering most extensively used, and perhaps the most successful one, is that utilizing specially prepared roofing papers. Those commonly used are made from ordinary deadening felt; they are run through a mill in which the felt is plunged into a number of successive baths of hot asphaltum and rolled hot and under pressure after each bath until the fibers of the paper are thoroughly impregnated with the asphaltum. Just before entering the last set of rolls, the paper is sprinkled with either mica or soapstone. These minerals, though no doubt adding somewhat to resistance against the attack of chemicals, have for their principal function the quality of keeping the paper from sticking in the roll in hot weather. Asbestos papers,

Mr. Bowie.

prepared in a manner somewhat similar, have also been used in a number of places. These papers are applied to the pipe by one of two methods known locally as the "spiral wrap" and the "longitudinal lap". The spiral wrap consists of applying the paper to the joints between collars by wrapping it around the pipe spirally. For this purpose, the paper is cut in the mill into rolls of the desired size, varying from 3 to 12 in. in width, and from 50 to 100 ft. in length, as best suits the diameter of pipe to be covered. The pipe is covered immediately after being screwed together by the tong gang, and while still on skids over the trench. It is coated with hot asphaltum, and the wider strips are wound on spirally before the asphaltum has had time to set. The crack which is left between each wrap of the wide strip is then coated with hot asphaltum, and a batten, or 3-in. strip, is wound on to cover it. The asphaltum is a thin grade of the ordinary refined product. It is applied to the pipe at a temperature of about 200° Fahr., and is of such consistency that it will not set for about 5 min. after application. If, due to climatic conditions or variation in the different shipments, the asphaltum sets too quickly, a so-called flux, which is simply a very thin grade of asphaltum, is added. It is usually the practice to ship 1 bbl. of flux to the field for every 60 bbl. of asphaltum. At the joints 3-in. strips stuck together with asphaltum are wound around the pipe on each side of the collar until a shoulder is built up flush with the outer circumference of the collar. The sleeve and shoulder thus built up are then coated with hot asphaltum, and the whole is covered with a 12-in. strip of paper. Besides being stuck to the pipe with asphaltum, this last strip is bound on with wire, as are also the ends of the paper where it is wrapped spirally. If this is not done, the ends will curl up and allow dirt to get between the paper and the pipe after the pipe has been lowered into the trench.

The longitudinal lap system, which, up to the present time, has been used much more extensively than the spiral wrap, consists (as the name implies) of wrapping the paper longitudinally around the pipe. The paper is delivered in the field in rolls, about 6 in. wider than the circumference of the pipe to be covered, each roll containing approximately 72 lin. ft. Two men go ahead of the paper gang and measure up each joint of pipe between collars and cut the paper into the desired lengths.

In applying the paper to the pipe undoubtedly the best method is first to coat the pipe with hot asphaltum, as previously described, then to wrap the sheet of paper around the pipe and stick down the lap with hot asphaltum. This, however, is not often done when covering lines where the oil passing through is to be heated, the supposition being that the heat of the oil (usually about 130° Fahr.), combined with the pressure of the earth on the pipe after it has been

buried, will be sufficient to soften whatever asphaltum there is in the paper and form the necessary bond between paper and pipe to keep the water out. Without question, this supposition is correct to a certain extent. The paper, especially where a heavy quality is used, will undoubtedly adhere to the pipe in most places where it comes in contact with the metal; but it is to be noted that roofing papers applied to a pipe, while the latter is still on skids above the ditch, and allowed to stand in the sun for a few hours, will stretch to such an extent that the contact will be found to be on the upper part only, there being often as much as ¼ in. of space between the bottom of the pipe and the paper. It is a serious question if this does not occur when the heat from the oil first reaches the paper, even though the pipe has been lowered into the trench and covered up. The paper, of course, does not bag uniformly, as in the case above ground, but the surplus is taken up in wrinkles, and these wrinkles are very apt to be connected for the entire length of a joint of pipe. Should an injury occur to one of these wrinkles or a crack develop in it, which will undoubtedly be the case as oxidation goes on, it is not at all improbable that water, which would enter, would reach every place in a joint of pipe where the paper was not securely stuck on, and thus render the whole covering practically useless. Where asphaltum is used as a binder, the wrinkles that form will be filled, thus preventing the water from entering, even though, in time, the paper should crack at these points. Time alone, however, can solve this much-mooted question, as there are no lines in California protected in this manner, which have been laid a sufficient length of time for these cracks to develop. Mr. Bowie.

For covering the collars, the same method is used as with the spiral wrap, except that the binder is often omitted. This, it would seem, is even more serious than omitting the binder from the main body of the pipe, as it gives the water a chance to get between the paper and the pipe through the ends of the wrinkles.

Whether or not a binder is used, the lap should always be made on the top of the pipe with the outer portion of the lap downward, and the wires should be placed about 16 in. apart. This is necessary in order to prevent the paper from being pulled away from the pipe by its own weight before it can be lowered into the ditch and covered.

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A METHOD OF DETERMINING STORM-WATER RUN-OFF.

Discussion.*

By Messrs. R. C. Strachan and Kenneth Allen.

Mr. Strachan.

R. C. Strachan, M. Am. Soc. C. E.—The work of the author in gathering and sifting data for comparing the results obtained from numerous run-off formulas, and in epitomizing the influences of the many factors in a new formula taking into account soil conditions, grades, rainfall, density of population, and extent of territory drained, is one of great magnitude, and is of permanent value to the designing engineer, even though, in his judgment, the new formula may not be universally applicable.

Such a formula is necessarily far from simple, and its practical utility must depend largely on the diagram or other device by which ease and celerity of solution are promoted.

It cannot be said that simplicity is the dominant feature of the diagrams accompanying the paper; and it is unfortunate that Mr. Buerger did not make use of the nomographic method of charting, by which Plate XIV could be reduced to three lines and the diagrams for finding P and N constructed so as to eliminate absolutely the maze of intersections shown in Figs. 3 and 4.

Mr. Allen.

Kenneth Allen, M. Am. Soc. C. E. (by letter).—The effort to find a run-off formula, both easy of application and reliable, is one that should meet with approval. If we could only be in possession of all the factors affecting the phenomena of run-off, its determination by a rational formula would leave nothing to be desired, and it has been the writer's opinion for some time that where conditions of topography or

* This discussion (of the paper by Charles B. Buerger, M. Am. Soc. C. E., published in August, 1914. *Proceedings*, and presented at the meeting of October 7th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Allen.

climate vary to a material degree from those on which such empirical formulas as the Bürkli-Ziegler are based, it would not be safe to use any but the rational method. The great difficulty here lies in the fact that the data necessary for its proper application—such as maximum intensity of precipitation and time of concentration—are so rarely known; and, if they are to be guessed at, why not use some more convenient empirical formula?

The weak point in formulas of the Bürkli-Ziegler type lies in their failure to provide for differences in the time of concentration due to the shape of the drainage area. They all have the merit of easy application, but, as the author points out, they were (excepting the Gregory formula) developed from local data, and, therefore, were not necessarily applicable to other places.

In any case, lack of care in the selection of proper values for C and R may result in large errors. In particular, the value of R appears to be subject to different interpretations by different engineers. It is usually stated—as by the author—as "rainfall, in inches per hour," but it is frequently taken as the rate of precipitation, in inches per hour for 10, 20, or 30 min., or for the probable time of concentration for the area in question. If we determine the rate of precipitation by Formula (13) this, and consequently Formula (9), will be twice as great if $t =$ 20 min. as if $t = 60$ min. The importance of definite understanding on this point is evident. The value to be assigned to R or I depends on the long-continued records of a self-registering gauge on or near the drainage district.

The determination of R for the conditions of maximum run-off, or I, involves the time for concentration, which in turn depends on the hydraulics of the drainage vehicle, the shape of the area drained, and the distribution of the rain on this area. These present a difficult problem to be solved for each sewer district, if accuracy is attempted, and the results of rain-gauge observations and sewer gaugings for long periods are necessary for its solution. Those who have attempted it know the care necessary to determine the correct discharge of a sewer in times of storm, where surface levels vary without apparent cause and friction coefficients can only be known by special experiments. One factor, which applies to large areas, that is often overlooked, is the decrease in average maximum intensity as the area increases. This factor has been called a "distribution coefficient" by Messrs. Wynne-Roberts and Brockmann,* who propose a value expressed by the formula, $C = 1 - 0.002 \sqrt{L}$, in which L is the length, in feet, of the longest line of sewer above.

The author reduces all values of R to that for 60 min., which he calls K, and then deduces his empirical formula to conform as well as may be with the results of actual gaugings. Besides the merit of being

* *Canadian Engineer*, Vol. 24.

Mr. Allen.

definite, in this respect, there is the further advantage in the use of K, pointed out by the author, that there are many more records available for 1 hour than for shorter periods.

Whether the relation of the maximum hourly rate to that for the shorter periods on which the capacity of sewers is usually based is constant for different locations seems to be improbable, and, for this reason, a formula using the shorter period, or one based on the time for concentration, would appear preferable.

One difficulty has been met in determining high rates of precipitation for short intervals with most of the recording gauges in use due to the necessarily small scale of hours ($\frac{1}{2}$ in. or less per hour). This has been ingeniously overcome by George A. Carpenter, M. Am. Soc. C. E., by causing a jar to be given to an arm carrying the recording pen every 5 min. by the clock mechanism. This produces a short horizontal line across the line on the chart which, at such times, approximates the vertical, and the precipitation for any 5-min. interval can then be scaled off quite correctly between any two consecutive horizontal lines.

The selection of K, or the frequency with which surcharge of sewers may be allowed, is an important and delicate matter, involving undue cost of construction on the one hand and the possibility of loss and consequent damage suits from flooding basements on the other. The assumption of a value of K corresponding to 12 months suggested by Dr. Imhoff as reasonable is, in the writer's opinion, far too short a period for well-developed conditions in large cities, where 10 years or more would often be warranted.

The writer has not examined the results obtained by the proposed formula sufficiently to express an opinion on its reliability under different conditions. It is admittedly somewhat awkward of direct application, but this practical difficulty is overcome by the use of diagrams, which is a sensible procedure in almost any case. Referring to Plate XIII, giving the frequency of 1-hour rainfalls of different intensities, it may be well to mention that the New York curve is based on records of the greatest single intensity for each month of observation, and that other high intensities were not recorded. For this reason, it is probable that the frequencies shown by this curve are slightly below the actual.

The author is to be congratulated on having produced, after so careful a study, a formula free from some of the objections found in those proposed heretofore.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES LEWIS FRAZIER, M. Am. Soc. C. E.*

DIED FEBRUARY 28TH, 1914.

James Lewis Frazier was born at Oakenwold, the fine estate of his father, at Staunton, Va., on June 17th, 1849. He was descended from the Scotch-Irish of the Shenandoah Valley, of Virginia, on both sides of the house. His father was William Frazier, of Staunton, an alumnus of Yale College and the University of Virginia, whose forbears came to Virginia from Ulster, Ireland. His mother was Sue Massie Lewis, of the sixth line of descendants from John Lewis, founder of Augusta County and Staunton, and his wife, Margaret Lynn Lewis, the daughter of the Laird of Loch Lynn in Scotland. John Lewis also came to Virginia from Ulster, Ireland; his sons, Gen. Andrew Lewis, Col. Charles Lewis, and Col. William Lewis, were conspicuous in Colonial and Revolutionary wars. Mr. Frazier was descended from Col. William Lewis (who was called the "Civilizer of the Border" during Colonial times), and his beautiful wife, Anne Montgomery.

His early education was under the direction of Dr. Junius, a French tutor at Oakenwold. Later, he went to Washington College, at Lexington, Va., and was there prepared for the University of Virginia under Gen. Robert E. Lee, then in charge of the college; he was a classmate there of Mr. Julius Kruttschnitt. He received his engineering education at the University of Virginia, and was a classmate of the late Samuel Spencer, M. Am. Soc. C. E.

Mr. Frazier was engaged on the location and construction of the Alabama Great Southern Railway, and the Chesapeake and Ohio Railway in the early Seventies, and was later associated with the late L. G. F. Bouscaren, M. Am. Soc. C. E., in the construction of the Cincinnati Southern Railway. He was also connected with the location and construction of many other lines of railroad.

In 1882, he was made General Superintendent of the Chesapeake and Ohio Southwestern Railway, with headquarters at Louisville, Ky. It was there that he married Miss Caroline Robinson, in 1887. In 1889, he was sent by the late Collis P. Huntington, F. Am. Soc. C. E., to the Southern Pacific Railway, and, under Vice-President Towne, was Superintendent, at various times, of most of the important divisions. At the request of one of the Directors of the Southern Pacific Railway, who was also interested in the Toledo, St. Louis and

*Memoir prepared by R. Montfort, M. Am. Soc. C. E.

Western Railway, he became General Manager of that road for two years. He was then made General Manager of the California Northern, and returned to San Francisco.

As the result of an attack of pleurisy while in San Francisco, Mr. Frazier was left with serious heart trouble and retired from active work. He spent the last nine years of his life traveling with his wife, mainly in Europe. He died in Rome, Italy, on February 28th, 1914, after a short illness. His remains were interred at Louisville, Ky., the former home of his wife.

He had no children, but, being of a sunny disposition, was especially fond of them.

Mr. Frazier was an accomplished engineer and a born leader, endowed with all the energy, honesty, and courage of his long line of Virginia ancestors. His modest, lovable disposition endeared him to his many friends and associates who will ever remember him with affection.

Mr. Frazier was elected a Member of the American Society of Civil Engineers, on September 1st, 1880, and served as a Director from the Pacific Coast District from 1902 to 1904.

JUSTUS HERBERT GRANT, M. Am. Soc. C. E.*

DIED AUGUST 1ST, 1914.

Justus Herbert Grant was born in Auburn, N. Y., on June 19th, 1849. His first engineering experience was obtained in 1866 and 1867, as Rodman on the preliminary and location surveys for the Southern Central Railroad.

In 1867, he entered the Sheffield Scientific School of Yale University, from which he was graduated in the class of 1870 with the degree of Ph. B.

In 1871, he served as Levelman and Topographer on the surveys for the Utica, Chenango and Cortland Railroad, from Cortland to Otselic, N. Y., and during a part of 1872, as Levelman and Assistant Engineer in charge of the construction of a short, narrow-gauge line, in Chenango County, N. Y., known as the Central Valley Railroad.

In 1872 and 1873, Mr. Grant was employed as Levelman on the preliminary surveys for the Auburn and Homer Midland, and Canandaigua, Palmyra and Ontario Railroads.

From 1874 to 1876, he was a member of the Engineer Corps of the New York Central and Hudson River Railroad, serving as Assistant Engineer in charge of earthwork and masonry construction

* Memoir prepared by Charles H. Grant, Esq., supplemented by information on file at the Society House.

of 16 miles and of the track-laying and ballasting of 32 miles of the line between Syracuse and Rochester, N. Y.

In the Fall of 1876, he resigned to accept a position as Engineer with the firm of George H. Thompson and Company, Contractors, of Rochester, N. Y., and as such was in charge of laying out and superintending the construction of all the work of that firm. In 1885, on the death of Mr. G. H. Thompson, the firm was dissolved, and Mr. Grant associated himself with the late H. M. Ellsworth, as the firm of Ellsworth and Grant, Contractors. This firm built many of the largest buildings in the city, including the Chamber of Commerce, the new Central Church, the Andrew Street Bridge, and the West High School.

In 1900 and 1901, Mr. Grant served as Commissioner of Public Works of Rochester, and in 1904 he was appointed Assistant City Engineer, in which capacity he served the city until his death on August 1st, 1914.

Mr. Grant was one of the most solid and influential business men of Rochester, of which city he had been a resident since 1874. His record and reputation in his profession were of the highest order, and he was regarded by his associates as one of the distinguished engineers of the United States.

He was a Trustee of the Mechanics Institute, and one of the founders of the Rochester Builders' Exchange, having served as its Secretary for many years. He was also a Director of the National Association of Builders; a member of the Permanent Arbitration Board of the Builders' Exchange; a member of the Brick Layers', Plasterers', and Stone Masons' Union; a Trustee of the Rochester Home for the Friendless; a Trustee of the Unitarian Church; and a member of the University Club of Rochester.

Although the community apparently has lost a devoted servant, a citizen of public spirit and of pride in its progress and advancement, his presence will be always felt, his exemplary life will be always an inspiration to those who knew him and loved him, and his work will go on.

Mr. Grant was elected a Member of the American Society of Civil Engineers, on March 2d, 1892.

DAVID ERNEST MELLISS, M. Am. Soc. C. E.*

DIED MARCH 24TH, 1913.

David Ernest Melliss was born in New York City, on March 11th, 1848. His professional education was received at the School of Mines of Columbia College, in that city, where he pursued the regular course

* Memoir prepared by Bolton B. Melliss, Esq.

in Mining Engineering for three years. Early in 1867, he matriculated at the University of Goettingen, in Hanover, Germany, and, after two years, which were chiefly devoted to chemistry, physics, geology, and mineralogy, he was graduated with the degree of Doctor of Philosophy in 1869. The following year, he attended lectures in the Natural Science Course at the University of Vienna. He was afterward constantly engaged in professional work, chiefly as a Consulting Civil and Mining Engineer, in the course of which he examined professionally and reported on mineral lands in the United States, Europe, Mexico, and Central America.

In 1873, Mr. Melliss was in charge of a topographical and geological survey of the Provinces of Guanacasta and Nicoya, for the Costa Rica Government. In 1881, he was appointed Consulting Engineer to the Pacific Gas and Electric Company, of San Francisco, Cal., and in its interest visited the principal gas-works of the United States, England, France, and Belgium, for the purpose of studying and reporting on the different processes.

The plans for the buildings and wharves of the Union Iron Works of San Francisco were designed by him and carried out under his supervision, as were also those for the Arctic Oil Works, which were built of concrete masonry throughout and completed in 1884.

The water-works of Mazatlan, Mexico, with its pumping plant, two double reservoirs, 22 miles of steel mains, and the entire city distributing system, were designed by Mr. Melliss, and constructed under his supervision from the inception of the enterprise to its completion in 1890. For several years he acted as Consulting Engineer to the Guadelupe de los Reyes Mining Company, the most important mining concern in Sinaloa, and he was also the Administrator of the San José de las Bocas Mines.

He was also engaged in considerable work in Venezuela, and, in recognition of certain professional services rendered that Government, was decorated with the Order of the Bust of Bolivar.

While in Mazatlan, in 1899, he complied with the request of the United States Minister to Guatemala, Mr. Mizner, to accompany him and act as Secretary of Legation during his visit to Nicaragua and Costa Rica.

Mr. Melliss' next important engineering work was the plant of the Olympic Salt Water Company, of San Francisco, representing an investment of about $500 000, every detail of which was designed and erected under his supervision.

In 1895, he acted as Superintendent of Works of the New City Hall Commissioners of the City of San Francisco, and at the same time followed his profession as Consulting Engineer. In this capacity (as Engineer), he remained until about 1904, when the pressing need of greater attention to his own affairs induced him to withdraw from

active life at a comparatively early age. In this manner, he spent the remaining years of his life.

In the middle of March, 1913, he was stricken with pneumonia, from which he never recovered, his death occurring at Mill Valley, Cal., on March 24th.

Mr. Melliss was elected a Member of the American Society of Civil Engineers on October 2d, 1895.

ARTHUR LOUIS PHILLIPS, M. Am. Soc. C. E.*

DIED JULY 13TH, 1914.

Arthur Louis Phillips was born in DuPage County, Illinois, on April 8th, 1864. He was educated at St. Ignatius College, Chicago, from which he was graduated in 1884. He immediately entered the service of the Chicago, Burlington and Quincy Railway, as a Rodman on construction in Missouri and Minnesota, and remained with that Company for two years. From 1886 until 1889 he was employed by the Northern Pacific Railway as Leveler on surveys in Montana and Washington. He was then promoted to the position of Resident Engineer. In 1889, he was appointed an Assistant Engineer on maintenance of way on the Southern Pacific Railway, with headquarters at Houston, Tex. In 1890-91-92 he was in the employ of the Denver and Rio Grande Railway as Resident Engineer and Locating Engineer.

On the opening of the Cherokee Strip, in Oklahoma, in 1893, Mr. Phillips was one of the first engineers in the field, and located many miles of the railway now forming parts of the Frisco System, being engaged on this work until July, 1899, when he entered the service of the Mexican Central Railway as Locating Engineer. He remained in that position until November, 1902, when he returned to Oklahoma and until March, 1903, was engaged on the location and construction of the Fort Smith and Western Railway.

In 1903 Mr. Phillips returned to the employ of the St. Louis and San Francisco Railroad, remaining with that Company until 1908, during which time he made surveys for the proposed extensions from Memphis to New Orleans, and was in charge of construction of the Frisco terminals in the latter city. He attained the position of Assistant Chief Engineer of the Frisco Railroad during this time, and made two trips to Mexico for the purpose of reconnoitering a proposed extension of the St. Louis, Brownsville and Mexico line from Brownsville to Vera Cruz. He was also sent to South America by some New York interests to look over a proposed railway in Colombia, which, however, was not built.

* Memoir prepared by F. G. Jonah, M. Am. Soc. C. E., and Clinton H. Fisk, Assoc. M. Am. Soc. C. E.

From November, 1908, until March, 1912, Mr. Phillips was engaged in private practice as Consulting Engineer in New Orleans, La. He then accepted a position as Chief Engineer of The Cuba Railroad, with headquarters at Camaguay, Cuba. He remained with this Company until June, 1913, and then resigned to accept a position as Assistant Chief Engineer of the Tela Railroad Company, in Honduras. On the completion of the location surveys for that work, he returned to New Orleans, and in December, 1913, reopened an office as Consulting Engineer. Shortly thereafter his health began to fail, and the last few months of his life were spent at various health resorts, but he declined rapidly, his death occurring in Denver, Colo., on July 13th, 1914. The body was taken back to New Orleans for interment.

On September 30th, 1905, Mr. Phillips was married, in Chicago, Ill., to Miss Jessie Fleming, who survives him.

He was a most courteous and affable gentleman, very considerate to, and greatly beloved by, his subordinates. He was a splendid Spanish scholar, and his experience in Latin-American countries had enabled him to speak the language fluently.

Mr. Phillips was elected a Member of the American Society of Civil Engineers on November 4th, 1903. He was a Thirty-second degree Mason and a Shriner, and was also a Member of the American Railway Engineering Association.

HORACE ARTHUR COOK, Assoc. M. Am. Soc. C. E.*

DIED MAY 17TH, 1914.

Horace Arthur Cook was born in Miller, Ind., on April 4th, 1877. He was graduated from Valparaiso University, in 1896, with the degree of B. S., and in the Fall of that year began work as Rodman on road construction in Lake County, Indiana. From September, 1897, to September, 1901, he was employed as telegraph operator, cashier, and agent for the Baltimore and Ohio Railroad, in Ohio and Indiana, and for the Northern Pacific Railway, in Washington.

In September, 1901, he entered Purdue University, from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1904. In June, 1909, he received the degree of Civil Engineer from the same University.

From August, 1904, to July, 1905, he was employed as Rodman, Field Draftsman, and Transitman, respectively, on the Galena and Wisconsin Divisions of the Chicago and Northwestern Railway, and from July to November, 1905, he served as Transitman, and Assistant Engineer in charge of 18 miles of construction, on the Wyoming and

* Memoir prepared by O. P. M. Goss, Assoc. Am. Soc. C. E.

Northwestern Railway. In December, 1905, he resigned his position with the Wyoming and Northwestern Railway Company to go to Seattle, Wash., where he was engaged, as Field Engineer for the Seattle Engineering Company, on various projects and surveys for hydro-electric developments, and in February, 1907, he was made a member of the firm and Vice-President of the Company.

In October, 1908, Mr. Cook's health failed, and he was forced to abandon his Seattle practice and move to Phœnix, Ariz., where he again engaged in general engineering work. He organized the Phœnix Engineering Company, of which he was President, and built a number of structures. Later, he made a specialty of road construction, in which branch of the profession he attained marked success and worked up quite an extensive consulting practice.

In 1906, he was married to Cora May Caulkins, of Lafayette, Ind., who, with one daughter, survives him.

Mr. Cook was a man of fine character, and his opinions and conclusions were highly respected by other members of the Profession. He had an exceptionally pleasing personality and was loved by all who knew him.

While at college, he took an active part in athletics, making a good record in the pole vault. He belonged to the Tau Beta Pi Fraternity and to the Order of Masons, and was also a member of the Pacific Northwest Society of Engineers.

Mr. Cook was elected an Associate Member of the American Society of Civil Engineers, on May 3d, 1910.

WILLIAM DE HERTBURNE WASHINGTON, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 30TH, 1914.

William de Hertburne Washington was born at Clover Sea, Hanover County, Virginia, on June 29th, 1863. He was the son of Lewis W. and Ella Basset Washington, and was a descendant of Charles, a brother of George Washington.

Mr. Washington was educated at Mary Schools and spent a term at college in special technical and mathematical study. He was to have entered the United States Navy, but no vacancy occurring at the time, he secured employment, on March 1st, 1880, on a preliminary survey for The Coal and Iron Railroad, in West Virginia, attaining the position of Instrumentman. The following year he was engaged as Assistant Engineer on the West Virginia Central and Pittsburgh Railway, and spent another year or more getting out timber and prospecting for ores and minerals on the lines of the Norfolk and Western and Chesapeake and Ohio Railways.

* Memoir prepared by Ernest Abs-Hagen, Esq., supplemented by information on file at the Society House.

Mr. Washington then came to New York City and was employed for some time in the office of the Chief Engineer of the American Atlantic and Pacific Ship Canal Company (the original Nicaragua Canal Company), and in other mechanical engineering work. In October, 1885, he was appointed United States Consul at London, Ont., Canada, and, in addition to his official duties, was allowed to act as representative of the firms of Messrs. Andrew Brothers, and Ferris and Walladay, Water-works Engineers and Contractors, reporting on and negotiating for the construction of water-works or hydraulic plants at Toronto, St. Thomas, Chatham, etc.

On his return to New York City, Mr. Washington organized the Hydraulic Contracting Company which built the water-works at Conneaut, Ohio, and hydraulic plants at many other places. When the firm was dissolved, he organized and became President and General Manager of the Hydraulic Construction Company which designed and constructed forty very important industrial plants in the United States.

After remaining in the construction business for about ten years, Mr. Washington went into the development of real estate, and at the time of his death, was President of the New Center Company, the Eastern Warehouse and Realty Company, etc.

In 1913, he was appointed Consulting Engineer to the Highway Commission of the State of New York, by Governor Sulzer, and was a Delegate to the Third International Road Congress, held that year in London, England. After the adjournment of the Congress, he visited various European countries for the purpose of inspecting roads and pavements, and on his return to the United States, made an elaborate report on his observations to the State Highway Commission.

Mr. Washington had recently published a book entitled "Progress and Prosperity," which embodied a comprehensive study of the mediums of development by the influence of railroad and highway transportation. He was also a contributor to the discussion on "Road Construction and Maintenance" before this Society in January, 1914.

He was the organizer and President of the West Virginia State Society, a member of the Sons of the Revolution, the Southern Society, the Virginia Historical Society, the American Society of Mechanical Engineers, the American Institute of Mining Engineers, and a Fellow of the National Geographical Society. He was also a member of, and until a short time before his death made his home at, the Calumet Club, in New York City.

Mr. Washington had just returned to New York City after a trip to Canada when he was stricken with cerebro-spinal meningitis, and died August 30th, 1914, after a short illness. His body was taken to Charlestown, W. Va., and interred in the family cemetery there.

He was unmarried, and is survived by a nephew and two sisters.

Mr. Washington was elected an Associate Member of the American Society of Civil Engineers on October 5th, 1892.

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"INVESTIGATION OF THE PERFORMANCE OF A REACTION TURBINE." R. L. DAUGHERTY.

"THE DIFFERENTIAL SURGE TANK." RAYMOND D. JOHNSON. (To be presented Dec. 2d, 1914.)

"PENSTOCK AND SURGE-TANK PROBLEMS." MINTON M. WARREN. (To be presented Dec. 2d, 1914.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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SUBMERGED PIPE WORK AT PORTLAND, OREGON.

By D. D. Clarke, M. Am. Soc. C. E.

To be Presented December 16th, 1914.

Synopsis.

In the following pages reference is made to the experience of the Water Department of the City of Portland, Ore., in supplying a portion of the city by means of several flexible-joint pipes laid under the bed of the Willamette River, a navigable stream passing through the central portion of the city.

The present gravity supply for the city is taken from the Bull Run River, a mountain stream within the limits of the Bull Run Reserve, which adjoins the Oregon National Forest, in the Cascade Mountains, and is brought thence to the eastern border of the city through two steel conduits 24 miles in length, one 33 to 42 in. in diameter, built in 1893 and 1894, and one 44 to 52 in. in diameter, completed in 1911.

The supply for the West Side or main business district is brought across the city through a 32-in. cast-iron main, the river being crossed by one 24-in. and one 30-in. flexible-joint steel main.

The object of this paper is to describe the pipes laid for the river section of this West Side conduit, the method of construction, and the cost, as well as the steps taken subsequently for the repair and reconstruction of the submerged pipes, as originally laid.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The work described covers:

(1) The laying of a 28-in. cast-iron, flexible-joint pipe in 1894.
(2) The laying of a 24-in. steel, flexible-joint pipe in 1898.
(3) The repair of the 28-in. cast-iron, flexible-joint pipe in 1909.
(4) The laying of a 30-in. steel, flexible-joint pipe in 1911.
(5) The work undertaken during 1913 and now just completed, consisting in taking up and relaying at greater depth the 24-in. line originally laid in 1898, and taking up and storing for future use at another point the 28-in. cast-iron pipe laid in 1894.

In further explanation of the paper, the writer desires to say that in making a search of engineering literature a few years ago he was able to find comparatively few references to flexible-joint pipes of the larger sizes, and therefore he has felt under obligation to place his experience within reach of engineers interested in this subject. For these reasons the following notes have been prepared and submitted.

28-Inch Line.—The pipe laid in 1894 (Plate XXX) consisted of 2 006 ft. of 28-in., cast-iron, flexible-joint pipe, 1½ in. thickness of shell, in net laying lengths of 15 ft. 8 in.

The contract price for furnishing this pipe was a lump sum for a certain specified length, which equalled	$31.4275	per lin. ft.
The contract for laying	2.50	" " "
Dredging, 18 746 cu. yd. at 40 cents	3.738	" " "
Total	$37.6655	per lin. ft.

or for 2 006 lin. ft. of 28-in., flexible-joint pipe, in place, a total of $75 556.

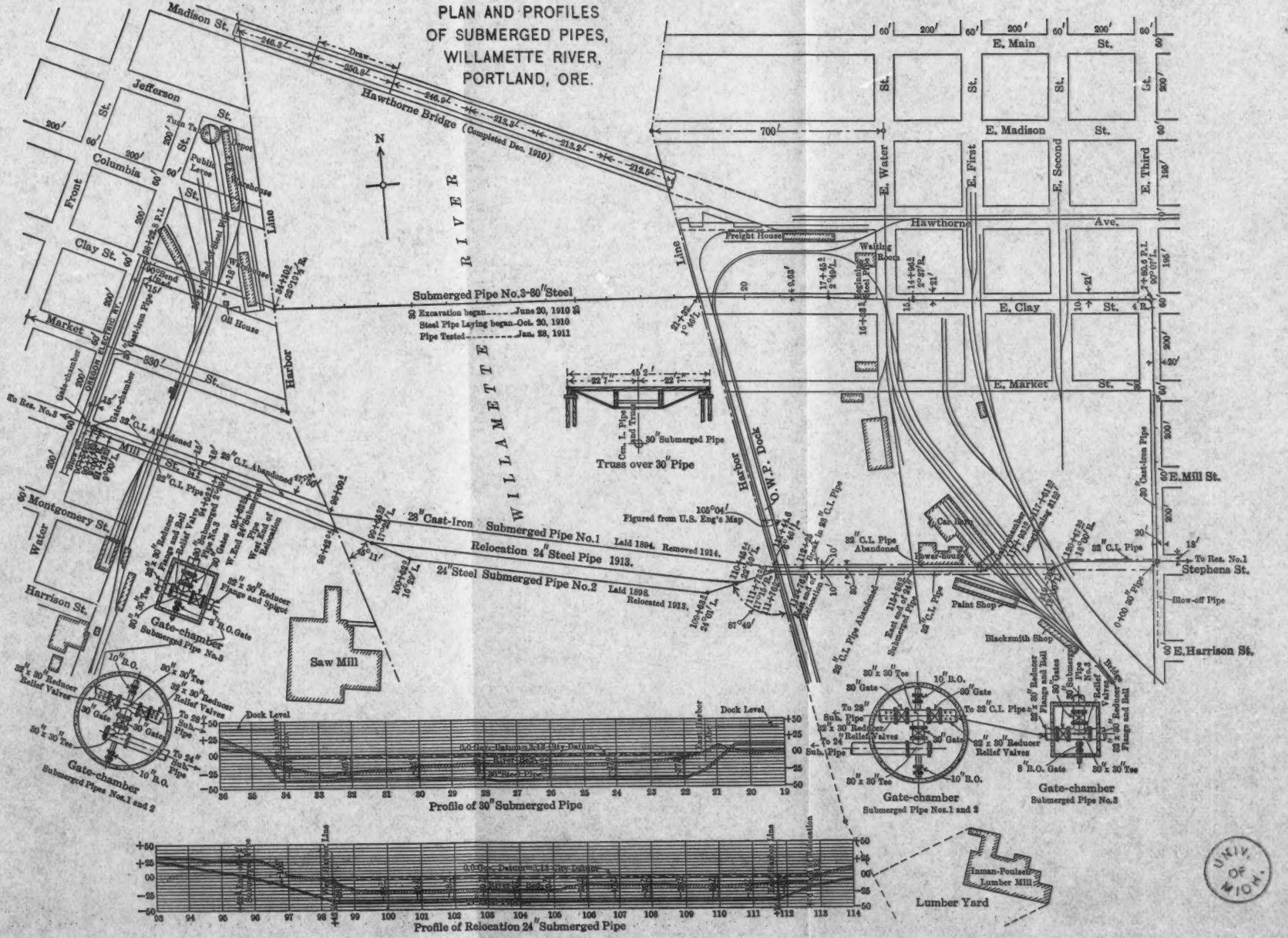
During 1894 the Water Department labor wage in Portland was $1.75 per day.

For details regarding this work reference is made to the paper* by Franklin Riffle and the late Albert S. Riffle, Members, Am. Soc. C. E., entitled "A Line of 28-In. Cast-Iron Submerged Pipes Across the Willamette River, at Portland, Ore." This paper and the discussions thereon are exceedingly interesting and instructive.

* *Transactions*, Am. Soc. C. E., Vol. XXXIII (1895), p. 257

PLATE XXX.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1914.
CLARKE ON
SUBMERGED PIPE WORK.

PLAN AND PROFILES OF SUBMERGED PIPES, WILLAMETTE RIVER, PORTLAND, ORE.



UNIV. OF MICH.

24-Inch Line.—The second pipe, the 24-in. submerged line laid in 1898, consisted of 2 041.6 lin. ft. of lap-welded steel pipe, ⅜-in. plate, including fifty-eight flexible joints, of special design, placed at intervals varying from 20 to 40 ft., depending on the alignment and grade of the pipe trench.

Messrs. Smyth and Howard, of Portland, Ore., were the general contractors for this work, the contract prices being as follows (including two joints and 72 lin. ft. of pipe not used):

60 flexible joints, $260.00 each	$15 600.00
1 938 lin. ft. of 24-in. pipe, $8.50 per lin. ft.	16 472.32
15 332 cu. yd. dredging, $0.40 per cu. yd.	6 132.80
2 041.6 lin. ft. pipe-laying, including joints, $1.77 per lin. ft.	3 613.63
Total	$41 818.75

During 1898 the Water Department labor wage in Portland was $1.75 to $2.00 per day.

This work was undertaken in the late summer, after the subsidence of the June freshet in the Columbia (which sometimes causes a rise of 20 ft. or higher at Portland), and was completed the same season.

The methods and appliances adopted for this work were similar to those adopted in laying the 28-in. line, 4 years before. A ladder-dredge was used in excavating the pipe trench, and the excavated material was taken away on barges to be utilized in filling low ground adjacent to the harbor line in another portion of the city. A cradle was also used in laying the pipe in the excavated trench, as described in the paper by Messrs. Franklin and Albert S. Riffle.

The flexible joint used (Fig. 1) was made in Portland, from designs prepared by the Water Department. Plate XXXI shows the cradle with which the pipe was laid.

The pipe, after it was laid, was tested in a manner similar to that adopted for the 28-in. line in 1894. The results secured did not indicate that the pipe was absolutely tight, but they were accepted at the time as being the best that could be obtained. The final test, under a pressure of from 170 to 180 lb., showed a leakage of approximately 7 gal. per min., or 10 080 gal. per day, or about 8 gal. per lin. ft. of joint, per day, under the pressures noted, which are approximately 50% in excess of the normal working pressure.

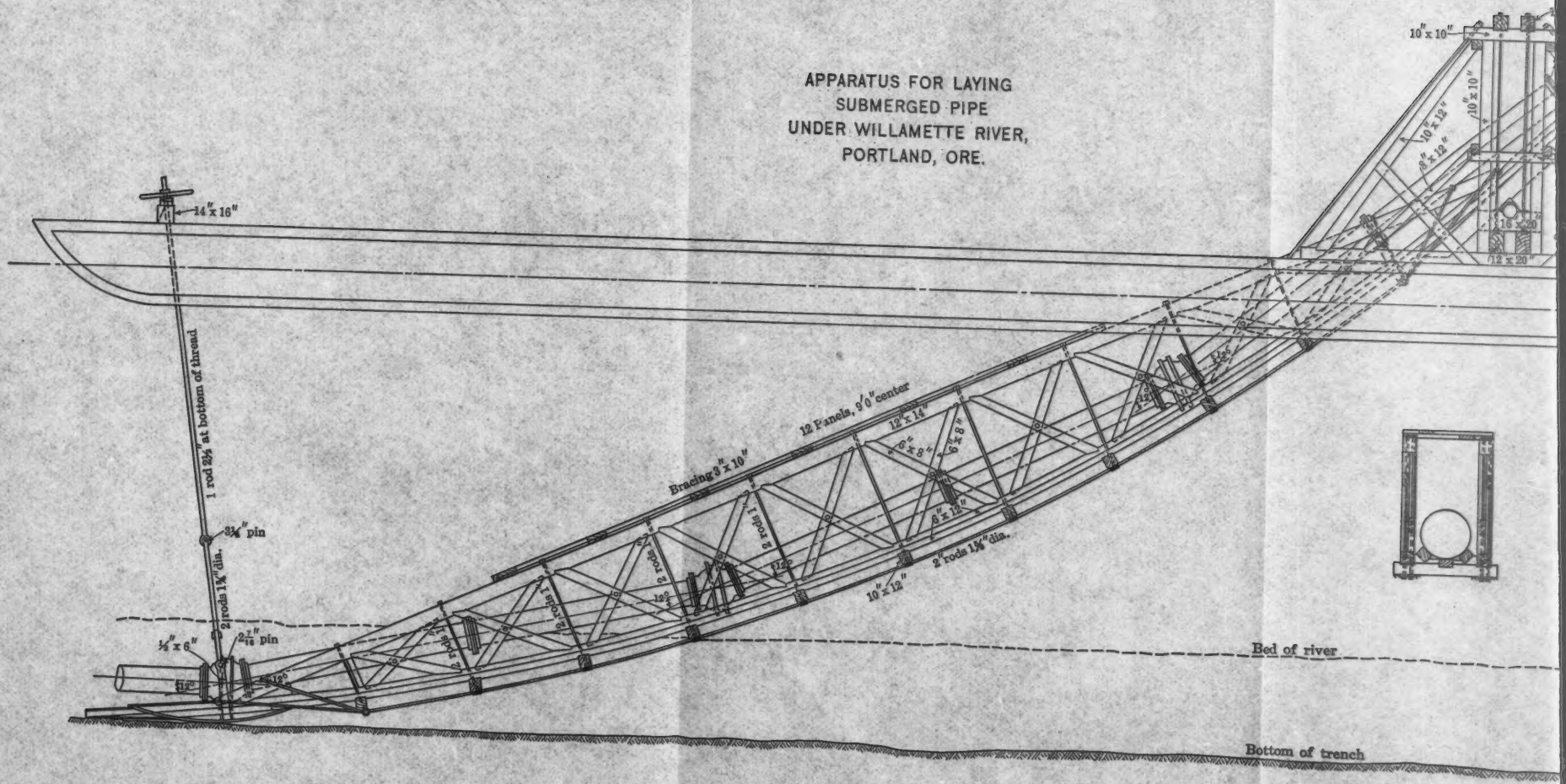
The location selected for this pipe (Fig. 2) was parallel to and approximately 10 ft. from the 28-in. line, until the harbor line was reached on each side of the river. The line was then deflected southward, or up stream, until a point was reached 100 ft. from the 28-in. line, which it paralleled for the remaining distance.

The profile of the original line shows a 22 to 24-ft. channel, 100 ft. or more in width, near the west shore, and, for the larger part of the distance thence to the east harbor line, the low-water depth did not exceed 12 to 16 ft. In establishing the grade for the original line, it was decided first to place the pipe in an open trench dredged to a depth of 8 ft. below the bed of the stream, a cover of 4 ft. being thus provided which was deemed ample to protect the pipe from injury by the passing or anchoring of vessels. In view of the existence of at least a 23 to 24-ft. channel adjacent to the west harbor line, which it was thought would provide for all shipping interests in that portion of the harbor (being at the head of deep-water navigation), it was not thought necessary to incur the extra expense required to dredge a trench to the depth of 30 ft. or more below low water for the entire distance between harbor lines. The matter was compromised, therefore, by placing the pipe grade so as to permit of dredging a channel at least 16 ft. in depth at all points within the harbor limits. This required dredging in places to a depth of from 12 to 14 ft. with at least an 8-ft. trench at all points. The original pipe line (the 28-in.) was laid on a grade established as just described, and the second pipe (the 24-in.) also conformed to the same grade, as nearly as practicable.

For each of the pipe lines heretofore mentioned the flexible-joint portion was a little more than 2 000 ft. in length (2 006 ft. for the 28-in. and 2 041.6 ft. for the 24-in. line). This length covered the distance from river bank to river bank and reaching a point where the grade of the pipe trench was above low-water level in the river. For several hundred feet additional on each side of the river, standard cast-iron pipe was laid to a connection with a concrete gate-chamber, where the two pipes were united with one 32-in. line forming a section of the main Bull Run Conduit for the supply of the West Portland District, the total distance between the east and west gate-chambers being 2 706 ft., and the approximate distance between harbor lines being 1 335 ft.

Repair of 28-Inch Line.—As has been stated, the 28-in. line was completed in 1894 and the 24-in. line in 1898. It was then thought

APPARATUS FOR LAYING
SUBMERGED PIPE
UNDER WILLAMETTE RIVER,
PORTLAND, ORE.



APPARATUS FOR LAYING
SUBMERGED PIPE
UNDER WILLAMETTE RIVER,
PORTLAND, ORE.

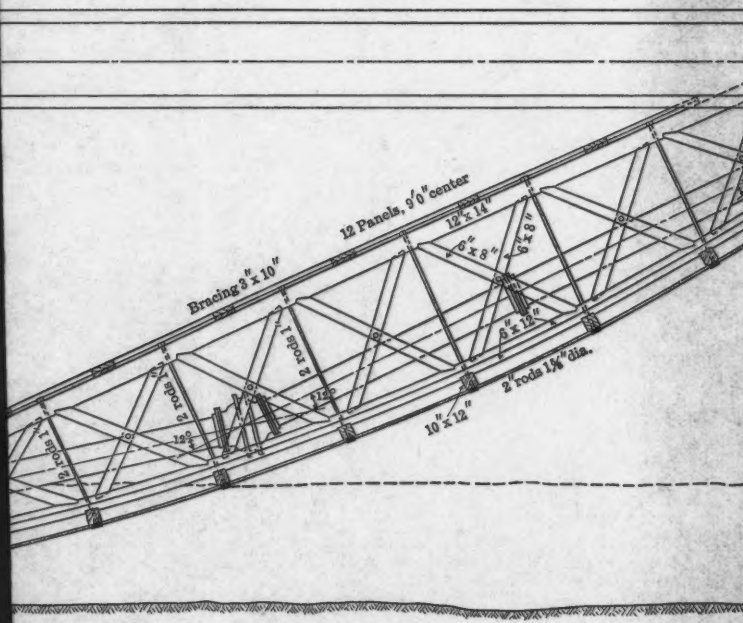
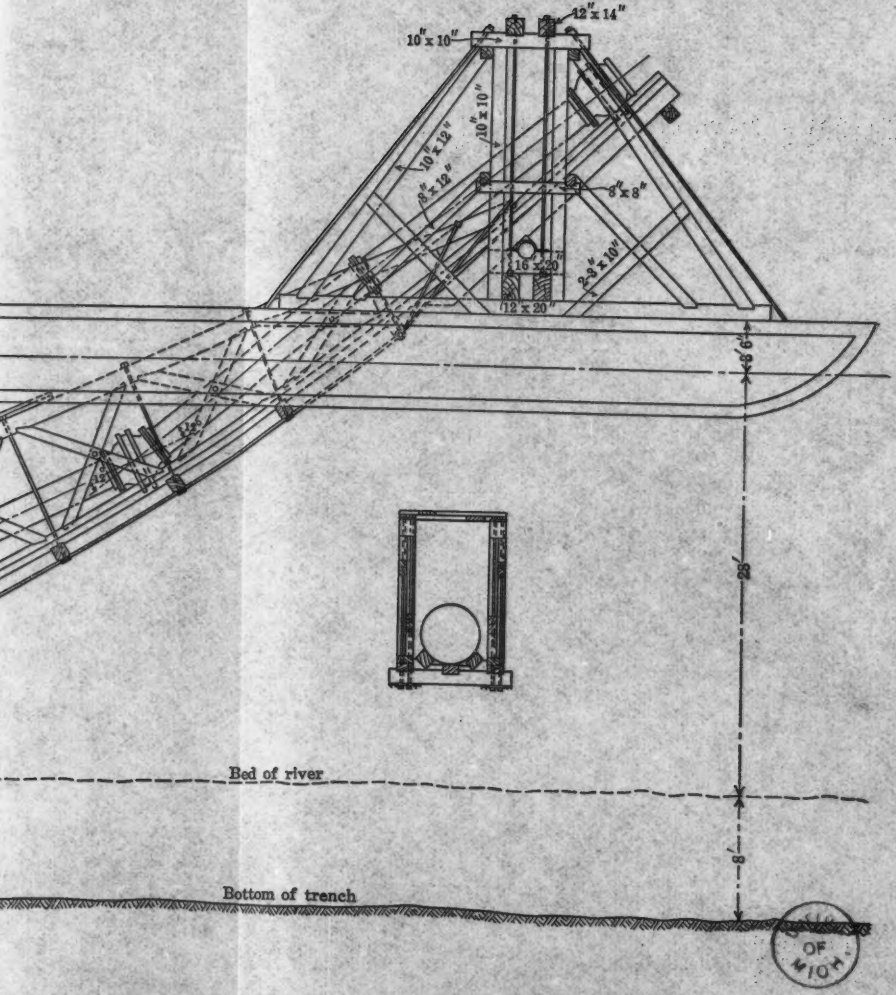


PLATE XXXI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1914.
CLARKE ON
SUBMERGED PIPE WORK.



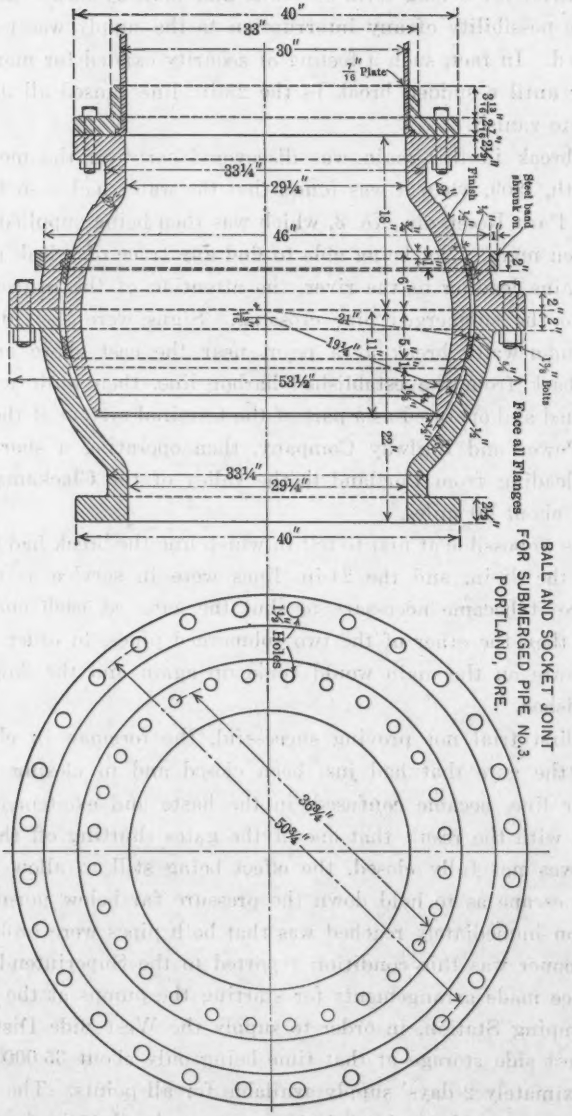


Fig. 1.

BALL AND SOCKET JOINT FOR SUBMERGED PIPE No. 3. PORTLAND, ORE.

that the double line would provide amply for all the needs of the West Side District for a long term of years, and that, by means of the two lines, the possibility of any interruption to the supply was practically eliminated. In fact, such a feeling of security existed for more than 9 years, or until a sudden break in the 28-in. line caused all dreams of security to vanish.

The break in this main was discovered early on the morning of April 10th, 1909, when it was found that the water had been falling in the City Park Reservoir, No. 3, which was then being supplied through the broken main. Not being able to find any trace of a leak along the street mains leading to the river, the attention of the inspectors was turned to the submerged pipe crossing. Signs were then discovered of an under-water break at a point near the east shore and about 100 ft. back from the established harbor line, then built up with a dock owned and operated as a part of the terminal system of the Oregon Water Power and Railway Company, then operating a short line of railway leading from Portland to the valley of the Clackamas, a distance of about 40 miles.

It was impossible at first to tell in which line the break had occurred, as both the 28-in. and the 24-in. lines were in service at the time. Therefore it became necessary to shut the gates at each end of first one and then the other of the two submerged pipes, in order to see if the pressure on the main would come up again and the flow become re-established.

The first trial not proving successful, the foreman in charge, in opening the gate that had just been closed and in closing those on the other line, became confused, in the haste and excitement of the moment, with the result that one of the gates shutting off the broken section was not fully closed, the effect being still to allow so much water to escape as to hold down the pressure far below normal. The conclusion immediately reached was that both pipes were broken.

No sooner was this condition reported to the Superintendent than he at once made arrangements for starting the pumps at the Palatine Hill Pumping Station, in order to supply the West Side District, the entire west side storage at that time being only about 35 000 000 gal., or approximately 2 days' supply available for all points. The Palatine pumps were started at about 10.00 A. M. on April 11th, for the first time since they were shut down on January 2d, 1895.

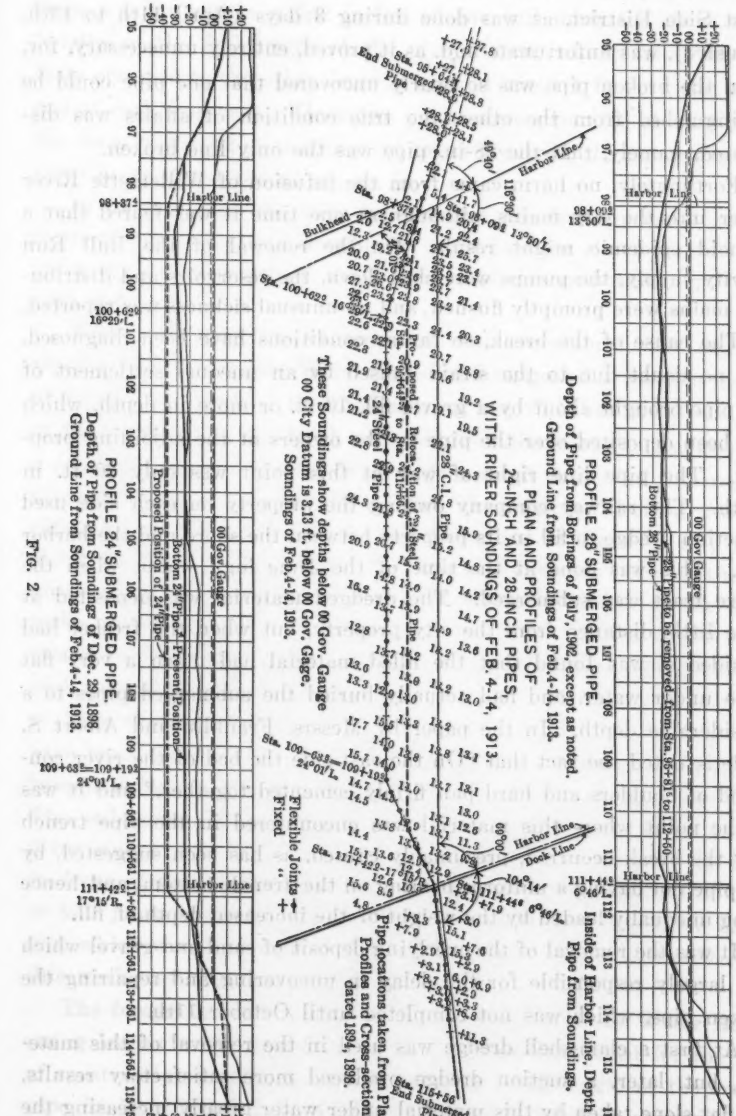

PROFILE 24" SUBMERGED PIPE.
Depth of Pipe from Soundings of Dec. 29, 1898.
Ground Line from Soundings of Feb. 4-14, 1913.
PROFILE 28" SUBMERGED PIPE.
Depth of Pipe from Borings of July, 1902, except as noted.
Ground Line from Soundings of Feb. 4-14, 1913.
PLAN AND PROFILES OF
24-INCH AND 28-INCH PIPES,
WITH RIVER SOUNDINGS OF FEB. 4-14, 1913.
These Soundings show depths below 00 Gov. Gauge
00 City Datum is 8.13 ft. below 00 Gov. Gauge.
Soundings of Feb. 4-14, 1913.
Flexible Joints
Fixed
Pipe locations taken from Plan,
Profiles and Cross-sections
dated 1894, 1898.
Inside of Harbor Line. Depth of
Pipe from Soundings.
Harbor Line
Dock Line
00 Gov. Gauge
Bottom 24" Pipe
Present Position of 24" Pipe
Proposed Position of 24" Pipe
Bottom 28" Pipe
28" Pipe to be removed from Sta. 95+81 to 112+60
Bulkhead
Sta. 95+00 End Submerged Pipe
Sta. 98+87
Sta. 100+622
Sta. 109+63.5=109+193
24°01'L.
Sta. 111+22
Sta. 111+44.6
6°46'L.
Sta. 115+56 End Submerged Pipe

FIG. 2.

The pumping of Willamette River water into the city mains in the West Side District, as was done during 3 days (April 11th to 13th, inclusive), was unfortunate and, as it proved, entirely unnecessary, for, when the broken pipe was so nearly uncovered that one pipe could be distinguished from the other, the true condition of affairs was discovered, namely, that the 28-in. pipe was the only line broken.

Fortunately, no harm came from the infusion of Willamette River water into the city mains, although at one time it was feared that a typhoid epidemic might result. On the renewal of the Bull Run gravity supply, the pumps were shut down, the reservoirs and distribution mains were promptly flushed, and no unusual sickness was reported.

The cause of the break, so far as conditions have been diagnosed, was no doubt due to the strain caused by an unequal settlement of the pipe brought about by a gravel fill, 16 ft. or more in depth, which had been deposited over the pipe by the owners of the adjoining property. The pipe line right of way at this point was only 30 ft. in width. The railway company owning the property on each side used a suction dredge to fill in its property between the shore and the harbor line. This was done at the time of the June high water, when the entire tract was submerged. The dredged material was deposited at some little distance from the city property, but when the freshet had subsided it was found that the filled material had taken a very flat slope under water, and had actually buried the submerged pipes to a considerable depth. In the paper by Messrs. Franklin and Albert S. Riffle is noted the fact that "On the east side the bed of the river consisted of boulders and hard pan firmly cemented together," and it was at the point where this material was encountered in the pipe trench that the break occurred, presumably caused, as has been suggested, by the pipe not having a uniform bearing on the trench bottom, and hence being unequally loaded by the weight of the increased depth of fill.

It was the removal of the overlying deposit of sand and gravel which was largely responsible for the delay in uncovering and repairing the broken pipe, which was not completed until October 11th.

At first a clam-shell dredge was used in the removal of this material, but, later, a suction dredge produced more satisfactory results, the flat slope taken by this material under water greatly increasing the volume of the excavation required. When the pipe was uncovered sufficiently to admit of examination by a diver, it was found that one of the 15 ft. 8-in. lengths of the 28-in. cast-iron pipe had been frac-

tured diagonally, the break being about 12 ft. in length and extending from a point within a few inches of one bell joint for about three-fourths of the full length of the pipe section, and crossing the pipe from one side to the other, leaving a point about 1 ft. in width at the end of each fragment. These two pieces were separated about 3 ft. by the force of the jet at the time the break occurred, the pressure at this point being about 150 lb. per sq. in.

In no case were the balls injured, but the fracture came so near to them that it was not possible to use a band or sleeve that did not cover the entire length of the fractured pipe, together with a ball at each end.

When the condition of the pipe was ascertained, the engineer reported the facts to the Water Board and suggested a possible method of repair. In view of the proximity of the 28-in. to the 24-in. pipe at the point where the break occurred, and the necessity for securing a continuous supply for the West Side without danger of interruption through the failure of the 24-in. line, the engineer decided that the safest plan would be to uncover the 28-in. line in the river at a point about 100 ft. from the east harbor line (the distance between the two pipes being approximately 75 ft.), and there, by means of the diver, make an under-water connection with the main and lay a new main thence to the east shore of the river, where a junction could be made with the old line above water level.

This would have required about 400 ft. of new pipe, which could have been quickly made of steel plates in local shops, and could have been laid from a platform along the north line of the pipe line tract. The only work in connection therewith requiring the services of a diver would have been the making of the joint with the old main in front of the harbor line. It was planned so that the use of three or four joints would have been sufficient for laying that portion of the pipe which would have been below water level. The estimated cost of this work was approximately $9 500.

The foregoing did not meet with the approval of the Water Board, owing largely to the cost, and it was therefore abandoned for the plan of constructing a split sleeve and encasing a full length of the broken pipe in its original bed, including a flexible joint at each end, the divers giving assurances that they could make the necessary joints under water.

Owing to the fact that at the point where the break occurred the

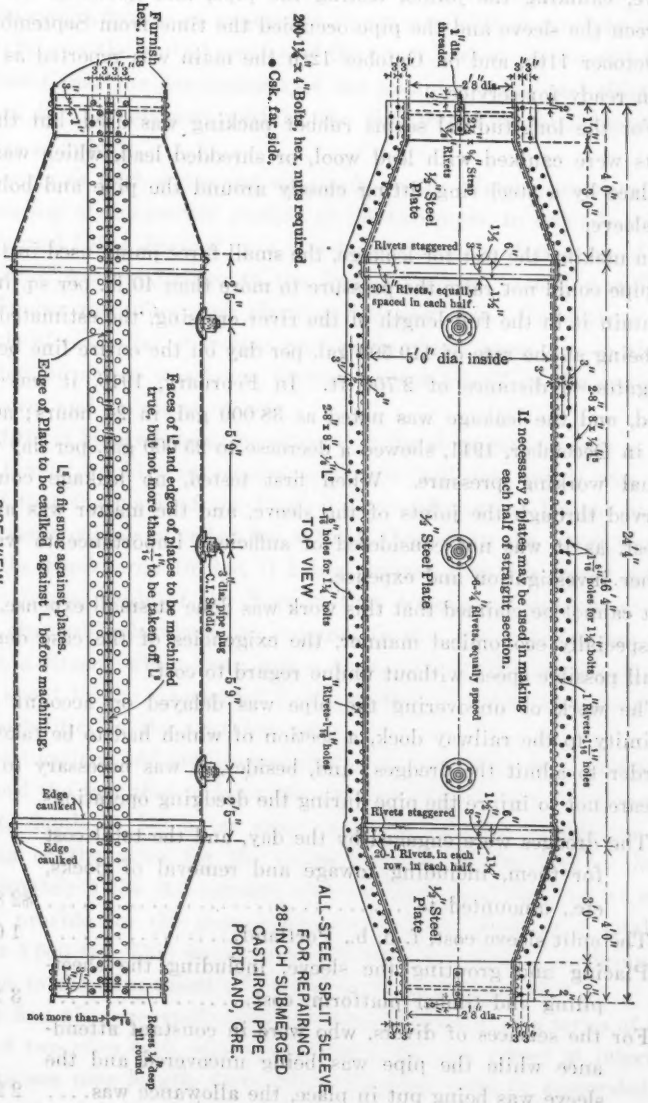
two pipes were near to one another (approximately only 2 ft. between them for a short distance) and that the broken pipe was also about 2 ft. lower than the 24-in. steel pipe which must be kept in service, it was determined that, prior to undertaking the repair work, and in order to protect the 24-in. main thoroughly, a row of timber sheet-piling should be driven between the two pipes, so as to avoid any danger of undermining the steel main by the excavation for the broken pipe. The pressure on this main being approximately 150 lb. at the bed of the river, it was decided to reduce this considerably before the piles were driven. This was done by shutting down the flow during the day, when the work of pile-driving was in progress, and turning it on again at night in order to maintain the supply in the City Park Reservoirs. One hundred and ten 9 by 12-in. "Wakefield" piles were driven, which occupied 5 days, and on the completion of this work, the additional dredging required to expose the 28-in. line fully was undertaken.

Preparatory to placing the split sleeve designed for this purpose (Fig. 3), the ends of the broken pipe were lifted partly from the position in which they were found, and also moved horizontally a short distance from the sheet-piling in order to give room for the sleeve. The fractured ends of the pipe were drawn together as closely as possible and then wired in place and afterward wrapped in heavy tarred canvas, securely fastened, the object of the latter being to prevent the cement grouting placed inside the sleeve from gaining admission to the interior of the pipe.

The sleeve was designed to be built of riveted steel plates, of sufficient diameter and length to take in an entire length of the pipe, with a flexible joint at each end. This meant that the main barrel of the sleeve must be about 4 ft. in diameter and 16 ft. long, with a reducer at each end, diminishing from 4 ft. to a diameter of about 32 in. at each end in a distance of about 3 ft., thus fitting the outside of the 28-in. pipe, making the extreme length over all 22 ft.

A contract for the construction of this sleeve was awarded to the Moran Company for $1 026.50, and it was placed by Department forces. The weight of the sleeve was 14 100 lb.

The progress of the work was delayed by the high stage of water due to the June flood in the Columbia River, and time was also required to secure the necessary materials and manufacture the sleeve. The sleeve was not received until September 8th, but the placing of it


Furnish hex. nuts
200 1¼" x 4" Bolts, hex. nuts required.
• Cak. far side.
Edge of Plate to be caulked against L before machining.
L' to fit snug against plates.
Faces of L' and edges of plates to be machined true, but not more than 1/16" to be taken off L'
Edge caulked
Edge caulked
not more than 1/16"
Recess ¼" deep all round
SIDE VIEW
FIG. 3.
1" dia. threaded
2'8" dia.
Rivets
Strap
¼" Steel Plate
Rivets staggered
20-1" Rivets, equally spaced in each half.
5'0" dia. inside
2-8" x 8" x ⅜" L
1⅛" holes for 1¼" bolts
TOP VIEW
If necessary 2 plates may be used in making each half of straight section.
⅜" Steel Plate
2-¾" Rivets, equally spaced
1" Rivets-1⅛" holes
20-1" Rivets, in each row, in each half.
24'4"
16'4"
4'0"
3'0"
2'6"
5'0"
5'9"
⅞" dia. pipe plug
C.I. Saddle
ALL-STEEL SPLIT SLEEVE FOR REPAIRING 28-INCH SUBMERGED CAST-IRON PIPE PORTLAND, ORE.

was commenced immediately thereafter. The work of placing the sleeve, caulking the joints, testing the pipe, and grouting the space between the sleeve and the pipe occupied the time from September 8th to October 11th, and on October 12th the main was reported as being again ready for service.

For the longitudinal seams rubber packing was used, but the end joints were caulked with lead wool, or shredded lead, which was held in place by a steel ring fitting closely around the pipe and bolted to the sleeve.

In making the test for leakage, the small force pump used in testing the pipe could not raise the pressure to more than 40 lb. per sq. in. and maintain it in the full length of the river crossing, the estimated leakage being at the rate of 119 500 gal. per day on the entire line between the gates—a distance of 2 706 ft. In February, 1910, it was again tested, and the leakage was noted as 38 000 gal. in 24 hours; a third test, in December, 1911, showed a decrease to 25 000 gal. per day under normal working pressure. When first tested, no leakage could be observed through the joints of the sleeve, and the matter was allowed to rest, as it was not considered of sufficient importance to warrant further investigation and expense.

It cannot be claimed that this work was done at small expense, or in an especially economical manner, the exigencies of the case demanding all possible speed without undue regard to cost.

The work of uncovering the pipe was delayed on account of its proximity to the railway dock, a section of which had to be taken out in order to admit the dredges; and, besides, it was necessary to exercise care not to injure the pipe during the dredging operations.

The dredges were engaged by the day, and the total cost for them, including towage and removal of docks, etc., amounted to	$2 850.00
The split sleeve cost, f. o. b., Portland	1 026.50
Placing and grouting the sleeve, including the sheet-piling and timber platform, cost	3 138.00
For the services of divers, who were in constant attendance while the pipe was being uncovered and the sleeve was being put in place, the allowance was	2 250.00
The total cost of the repairs, including the items just mentioned and incidental expenses, was	$10 750.00

Lowering the Pipes.—As already stated, the grade for the first pipes laid was established so as to provide for an available channel depth of only 16 ft. over the central portion of the upper harbor. It was not then anticipated that the development of the manufacturing and shipping interests in the southern part of the city would demand increased facilities so quickly.

As early as 1902, a demand was made on the Water Committee for the lowering of the central portion of the two pipes, in order to admit of a channel being dredged on a direct line from the draw-span of the Hawthorne Avenue Bridge to the Inman-Poulsen Company mill—a large lumber manufacturing plant on the east side. Complaints and demands of a similar character were made nearly every season thereafter, but no action was taken thereon, except to call on the engineer for reports and estimates of cost.

In 1911 it finally became manifest that the shipping interests in the upper harbor would soon require improved facilities, and it was also learned that the plans of the Port of Portland and the United States Government engineers called for a 30-ft. channel in the upper harbor above the Hawthorne Bridge, it being claimed that this would suffice for the future needs of the Port in that direction, regardless of the depths that will ultimately be required in the lower Willamette and Columbia Rivers, where even now a 40-ft. channel is dreamed of and talked about by the commercial bodies of the city and State as among the possibilities of the next few years.

Investigations and studies undertaken in connection with this work developed the fact that to lower the two submerged pipes so as to give a 30-ft. depth at low-water stage would practically necessitate the lowering of the pipes for the entire distance between harbor lines, approximately 1 335 ft., together with an additional distance at each end to provide for the approaches to the lower level, or a total of at least 1 500 ft. This would involve a large quantity of dredging, in addition to the adjustment of the pipes in the new trench.

The first plan prepared called for the construction of a series of pile bents of two piles each, with timber caps and ties, placed at intervals equal to one pipe length, from which the pipe could be suspended by slings, and rods with threads of sufficient length to permit the pipe to be lowered into the new position after the supporting earth had been removed and the new trench excavated. It was thought that the

necessary excavation could be made from the side of the pipe with a suction dredge which could be manipulated so as to undermine the pipe gradually and admit of lowering it.

This plan was abandoned later, owing, in part, to the difficulty in handling the dredged material which had to be barged away and disposed of outside of the river channel.

The method finally adopted called for taking up the two pipes and, after making necessary repairs, relaying them in the trench excavated to the required level, as described later.

30-Inch Line.—After considering the difficulties attending the lowering of the two pipes and the danger of breaking one or both of them, and the consequent interruption of the supply for the entire West Side, or main business district of the city, the Water Board decided that, in the interests of safety, a new and entirely separate submerged pipe line should be laid in advance of any work which might in any manner disturb the two pipes then in use. Plans and specifications were therefore prepared by the office staff calling for a 30-in. line crossing the river at the foot of Clay Street, or approximately 600 ft. distant from the two pipes already in use.

The new line was planned to leave the 32-in. supply main at a point about 400 ft. east of the original East Side gate-chamber at 3d and Stephens Streets, and to connect again with the 32-in. main at the West Side gate-chamber at Water and Mill Streets.

This plan called for 4 312.6 ft. of entirely new pipe. Of this, 2 265.6 ft. on the east and west sides were to be standard, 30-in., cast-iron pipe, Class "F", New England Water Works Association specifications, and the remaining 2 047 ft. were to be 30-in., lap-welded, galvanized and asphalted steel pipe, of $\frac{7}{16}$-in. plate with flange joints. Inserted in the steel main, at intervals of about 40 ft., were placed twenty-seven flexible joints of cast iron.

The grade established for the pipe trench between the harbor lines was 38 ft. below low-water level, which provided for a 3 to 4-ft. cover over the top of the pipe, with a 30-ft. depth of channel, as called for by the plans of the Port of Portland and the U. S. Engineers.

This work was commenced in August, 1910, and completed in March, 1911.

The dredging of the trench for this main was commenced by one of the suction dredges operated by the Port of Portland Commission,

at a rental of $250 per day. The work with this dredge was soon discontinued, however, as it was needed for channel work at another point, and the dredging was completed with the dipper-dredge owned by the Pacific Bridge Company, this Company being in a position to utilize the dredged material for filling purposes in a near-by district.

The 30-in., standard, cast-iron pipe used for the shore connections was manufactured by the Oregon Iron and Steel Company, at its foundry near Portland, the price, in place, being $11.10 per ft.

The flexible joints were made by the John Wood Iron Works, of Portland, from plans furnished by the Water Department, for $345 each.

The 30-in., lap-welded steel, galvanized and asphalted pipe was purchased from the National Tube Company (2 200 ft.) at $14 per ft.

The contract for laying the pipe and making the joints was awarded to Robert Wakefield and Company, of Portland, for the prices given in Table 1.

TABLE 1.—Cost of Pipe Laying.

Trench excavation	5 705 cu. yd. at	$0.90	$5 134.50
Cast-iron pipe in place	2 265.6 ft. "	11.10	25 148.16
Steel pipe, with flexible joints, laid	2 047 lin. ft. "	4.50	9 211.50
Ball-and-socket joints (only 27 used)	30 "	345.00	10 350.00
Port of Portland dredge, estimated cost of	102 655 cu. yd.		16 160.14
Pacific Bridge Company's dredge, estimated cost of	29 700 cu. yd.		4 158.00
Valves, 30-in.	4 at	$590.00	2 360.00
Concrete, in two gate-chambers	59 cu. yd. at	12.00	708.00
Specials	43 962 lb. at	0.06	2 637.72
Extras: labor and materials, dock repairs, etc.			10 197.84
Steel pipe used	2 047 ft.		28 658.00
Total			$114 723.86

Summary of Total Costs:

Steel pipe	$29 135
Dredging	20 318
Cast-iron pipe and pipe laying	65 747
Labor and engineering	4 868
Total	$120 008
Original estimate	125 000

On the completion of the work, the pipes, including the standard cast-iron and flexible-joint steel pipe, were tested to 175 lb. per sq. in., and the leakage under the normal pressure of 150 lb. was found to be approximately 30 000 gal. per day.

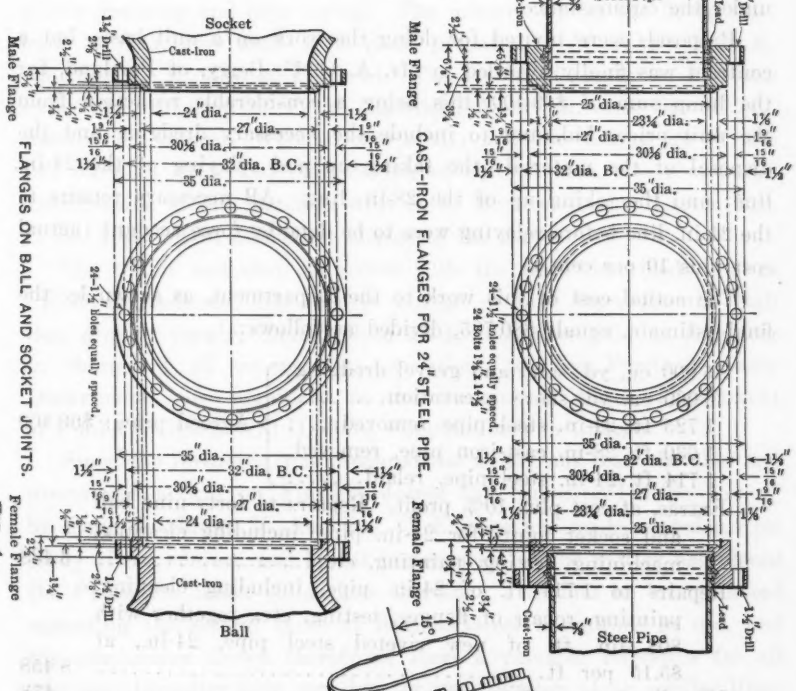
Having become satisfied that this loss was from no one leak of special size, but rather from a number of small leaks, no attempt was made to check them. It may be stated that the last test was made on September 28th, 1912, when the seepage loss was at the rate of 5 000 gal. per day—a decrease of 25 000 gal. per day from that which occurred shortly after the main was laid.

Lowering of 24-Inch Line.—Soon after the completion of the new line, in 1911, a renewed application was made by the milling interests and the Port of Portland Commission, for the removal of the original submerged pipe lines, which were claimed to be a serious obstruction to navigation in the upper harbor.

An attempt was made to induce the Port of Portland Commission to share in the expense of this work, at least to the extent of contributing the use of its dredges for the removal of the material which it would be necessary to excavate to uncover the pipe and prepare the new trench at a lower level, but the inability of the Water Department to secure a satisfactory dumping ground prevented the consummation of the arrangement finally agreed upon, namely, that the Port of Portland would deduct from the sum due for the use of its dredges (at $350 per day) a sum sufficient to cover the cost of dredging a channel 300 ft. wide at the submerged pipe crossing a distance of 100 to 200 ft. on the line which had been planned, extending from the draw-span toward the upper river channel. This comprised a very small part of the total work required for this channel, which had been laid out in such a position as to oblige the city to lower its pipes for the entire distance between harbor lines, the natural deep-water channel near the west harbor line having been entirely ignored.

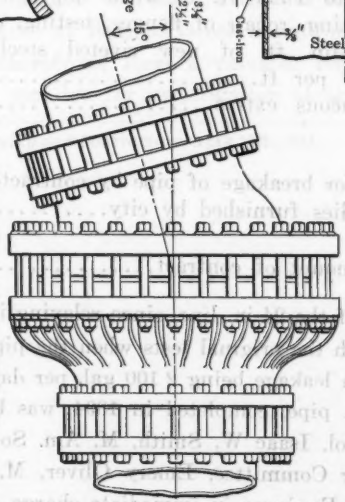
The conclusion of the whole matter was a decision of the Water Board to advertise for bids for the entire work, independent of any promise or expectation of aid from any source. It was furthermore decided to ask for proposals, on a unit basis, for the necessary dredging and the disposal of the dredged material; for uncovering and taking up both the 28- and 24-in. pipes; and for relaying the 24-in. pipe only—the 28-in. pipe to be stored for future use at some other crossing.

It was deemed that the two steel pipes, 30 and 24 in. in diameter, respectively, would provide an ample supply for the West Side District, the estimated capacity of the two pipes being nearly 40% in excess of that of the 32-in. line forming the shore ends of the submerged



FLANGES ON BALL AND SOCKET JOINTS.
Fig. 4.

CAST-IRON FLANGES FOR 24" STEEL PIPE.



BALL AND SOCKET JOINT ASSEMBLED.

DETAILS OF FLANGES
USED ON
24-INCH SUBMERGED STEEL PIPE.
PORTLAND, ORE.

pipes and connected with the city reservoirs, and therefore the relaying of the 28-in. cast-iron main would not be necessary or advisable under the circumstances.

Proposals were invited for doing the work on a unit basis, but a contract was finally awarded to Mr. A. C. U. Berry, of Portland, for the lump sum of $69 400, this being a considerable reduction from the unit prices bid, and to include the necessary dredging and the disposal of the material; the taking up and relaying of the 24-in. line; and the taking up of the 28-in. line. All necessary repairs to the 24-in. line before relaying were to be done by force account (actual cost plus 10 per cent.).

The actual cost of this work to the Department, as shown by the final estimate, equals $80 785, divided as follows:

85 000 cu. yd. sand and gravel dredging. 6 060 cu. yd. shore excavation..... 1 725 ft. 24-in. steel pipe removed..... 1 630 ft. 28-in. cast-iron pipe, removed. 1 714 ft. 24-in. steel pipe, relaid.....	Agreed price:	$69 400
Extras, at cost plus 10% profit. Repairs to forty-nine ball and socket joints for 24-in. pipe, including cleaning, machining, testing, painting, etc.....		3 438
Repairs to 1 714 ft. of 24-in. pipe, including cleaning, painting, repair of flanges, testing, etc., together with 808 lin. ft. of new riveted steel pipe, 24-in., at $5.15 per ft.....		8 458
Miscellaneous extras.....		478
		$81 774
Deduct for breakage of pipe by contractor, and for minor supplies furnished by city.....		989
Net amount of contract.....		$80 785

The test of the 24-in. line, since relaying in new position, compares favorably with the original tests when the pipe was first laid, in 1898, the maximum leakage being 2 100 gal. per day.

The 28-in. pipe, completed in 1894, was built under the direction of the late Col. Isaac W. Smith, M. Am. Soc. C. E., Chief Engineer of the Water Committee, Emery Oliver, M. Am. Soc. C. E., being the Assistant Engineer in immediate charge of the work for the city. The flexible joints used for the 24-in. line, completed in 1898, were

designed by the late J. A. Lesourd, mechanical engineer, of Portland, W. W. Amburn, M. Am. Soc. C. E., was the Assistant in charge of the dredging and pipe laying. The joints used for the 30-in. line, built in 1911, were made from plans prepared by W. R. Phillips, M. Am. Soc. C. E., also of Portland. The laying of the 30-in. line was under the direction of F. M. Randlett, Assoc. M. Am. Soc. C. E., Assistant Engineer, Water Department; and the taking up of the two pipes in 1913, and the relaying of one of them, was also under the direction of Mr. Randlett with B. S. Morrow, Assoc. M. Am. Soc. C. E., Assistant in immediate charge of the work.

The writer was also connected with the work described, first as Principal Assistant Engineer in charge of pipe lines for the Bull Run gravity supply, completed in 1894, and since 1897 as Engineer in charge of all construction operations for the Portland Water Department, now organized as the Bureau of Water, Department of Public Utilities, Will H. Daly, Commissioner in Charge.

Since the foregoing was prepared the writer has read the able and instructive article on Flexible Joints for Submerged Pipe Lines,* by Emil Kuichling, M. Am. Soc. C. E., who has made an exhaustive study of the subject, and by his presentation of the matter has placed the Profession under obligations therefor. The data he has gathered regarding the several types of flexible joints heretofore in use, and his conclusions drawn therefrom, form a valuable reference for all who may hereafter have occasion for investigation along similar lines.

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* *Engineering and Contracting*, April 15th, 1914.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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HISTORY OF LITTLE ROCK JUNCTION RAILWAY BRIDGE, OF THE ST. LOUIS, IRON MOUNTAIN AND SOUTHERN RAILWAY COMPANY, OVER THE ARKANSAS RIVER AT LITTLE ROCK, ARKANSAS, 1883-1914.

By C. E. Smith, M. Am. Soc. C. E.

To be Presented December 16th, 1914.

Synopsis.

Object.—This paper was written for the purpose of informing the Engineering Profession of the troubles encountered at the Little Rock Junction Railway Bridge, due to the defective construction of the piers, in order that such defective construction may be avoided in the future; its purpose is also to point out the methods suggested and used to accomplish the correction of the defects, in order to guide the Profession in the correction of similar defects which may cause trouble in future in similar piers.

Digest.—The bridge consists of three 253 ft. 4-in., through truss spans, one 127 ft. 11-in., through truss span, and one 352 ft. 10-in., through truss swing draw-span, all single-track, resting on masonry piers and one abutment. Four of the piers are supported on pneu-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

matic caissons and timber cribs. The pneumatic caissons are of ordinary timber construction, and rest on bed-rock, approximately 45 ft. below low-water line.

The timber cribs over the pneumatic caissons, extending up to low water, consist of 12 by 12-in. timbers, 3 ft. from center to center, sheeted outside with 3-in. planks. The spaces between the timbers were supposed to have been filled with rip-rap, but only a small quantity was placed, the greater number of the cavities having been filled with sand discharged from the caissons.

The caissons were poorly located, and when the working chambers were finally sealed, the tops of the timber cribs were considerably out of place. As the steel superstructure was on the ground at the time of the commencement of the masonry work, the masonry piers were located on top of the cribs in such a way that the coping courses were the proper distances apart to receive the steelwork, which made it necessary to place the piers at one side or the other of the timber cribs and use different batters.

The bed of the Arkansas River is composed of fine sand and silt, which is scoured to great depths during floods, especially around obstructions such as piers. The scouring of the river around these piers caused the sand to run out of the cribs through the spaces between the sheeting planks, and the timbers, being deprived of the support of the filling material, settled under the weight of the bridge and traffic.

As the masonry piers were not located concentrically with the cribs, the timbers compressed unequally and caused the piers to lean, their tops moving in various directions. The piers were a constant source of trouble and expense from the date of construction in 1883 until recently when they were reconstructed.

The reconstruction consisted in placing annular pneumatic caissons around the old piers, filling the spaces in the old cribs with concrete, enlarging the footing by filling in with concrete over the new annular caissons, and encasing the old piers in reinforced concrete following symmetrical lines, and enlarged for a double-track superstructure in the future.

The excessive scour of the Arkansas River renders practically impossible the maintenance of falsework for long periods, and swift run-outs have been experienced in all months. As the hazard attending

the maintenance is very great, the work was carried out without falsework.

The paper also describes in detail the various investigations to which this bridge was subjected by engineers, and the various unsuccessful attempts, made at great expense, to correct the trouble in advance of the final reconstruction.

Conclusions.—Timber cribs extending from pneumatic caissons to low-water level are entirely unsuitable for bridge pier foundations unless they are of solid timber construction, or, preferably, if the spaces between the timbers are carefully packed full of rock and thoroughly grouted, or, best of all, if the spaces are filled carefully with concrete and well rammed so as to fill all voids.

When defects are discovered in bridge piers, and trouble results, correction should be applied at the seat of the trouble; expenditures for temporary expedients are a waste of money. The continuation of the trouble without adequate correction involves constant hazard which can be avoided by efficient engineering talent.

Descriptive.

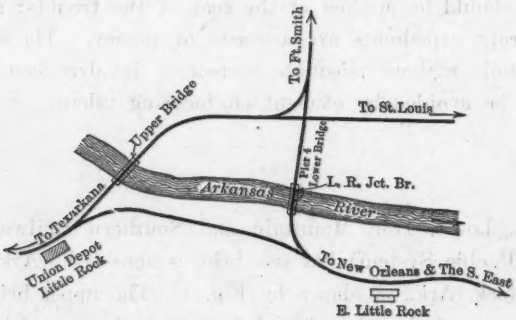
The St. Louis, Iron Mountain and Southern Railway Company (Missouri Pacific System) has two bridges across the Arkansas River at Little Rock, Ark., as shown by Fig. 1. The upper bridge, resting on pneumatic cylinder piers founded on rock, is crossed by main-line freight and passenger trains from points northeast of Little Rock to Texas and the Southwest. The lower bridge, founded on masonry piers, timber cribs, and pneumatic caissons on rock, is used by freight trains only from northeastern points to Louisiana and Southeastern Arkansas. Plate XXXII gives an elevation of this structure, commonly known as the Little Rock Junction Bridge. It consists of one 352 ft. 10-in. swing draw-span, three 253 ft. 4-in. simple truss spans, and one 127 ft. 11-in. simple truss span and trestle approach, all single-track, built in 1883. Fig. 2 is a view of the structure. The south abutment is of masonry built on the rock that outcrops on the south bank. The pivot pier, commonly known as Pier 2, and Piers 3, 4, and 5, are rock-faced, concrete-filled, masonry piers, about 45 ft. high, resting on filled timber cribs and pneumatic caissons about 40 ft. high, the masonry of the draw-pier being annular, with

a well down to the crib. Piers 6 and 7, are rock-faced, concrete-filled, masonry piers built on piles. The only specifications that can be found for the piers are as follows:

"The piers to rest on pneumatic caissons sunk to rock filled with concrete, timber cribbing to reach from roof of caissons to 4 ft. below low-water mark.

"This crib to be drift-bolted and planked on outside and filled with sand and stone; on this crib the masonry to be started and built up to grade line.

"Piers to be 6 ft. 6 in. wide under coping and 20 ft. long, with semicircular ends, with a batter of ½ in. per ft. on sides and up-stream ends, but no batter on down-stream ends; piers to consist of solid walls, averaging 2 ft. thick, built of dimension stone so as to make joints not exceeding ½ in.; inside space to be filled with concrete. This refers to three piers.



LOCATION OF LITTLE ROCK JUNCTION BRIDGE, LITTLE ROCK, ARK.

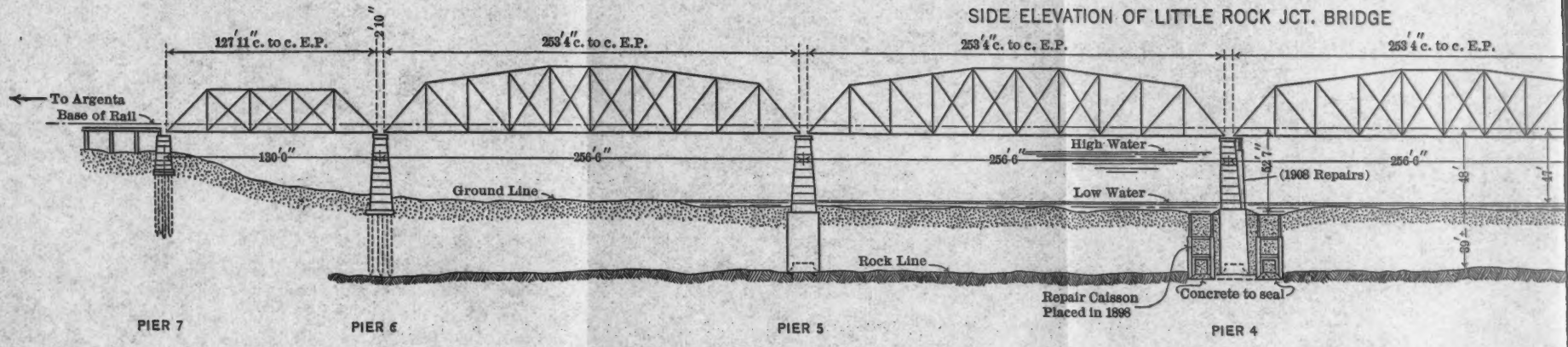
FIG. 1.

"Draw-pier to be 30 ft. in diameter under coping, with 8-ft. wall; center to be left open; pier to rest on caissons sunk to rock and filled with concrete with crib as before.

"Fifth pier on east bank to rest on piles sawed off and capped below low-water mark; to be 6 ft. 6 in. wide under coping and 20 ft. long, with semicircular ends, to have a batter of ½ in. per ft. on sides, but no batter on ends; in other respects to be the same as Piers 1, 2, and 3. All work to be done in a thorough workmanlike manner and to the satisfaction of the General Manager of the Little Rock Junction Railway.

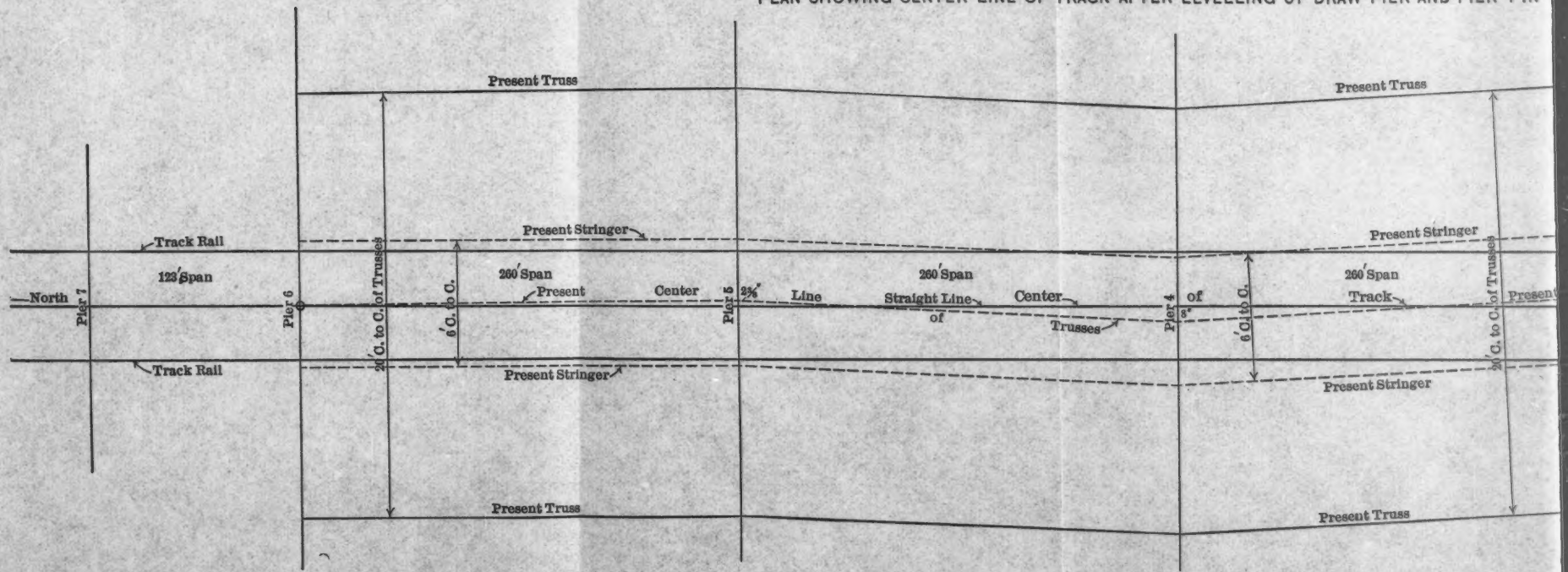
"The draw protection and draw-pier to be completed by the first day of June, 1884, and all the work embraced in this contract to be completed by the first day of September, 1884, unless some disaster by flood or by other uncontrollable cause shall prevent.

SIDE ELEVATION OF LITTLE ROCK JCT. BRIDGE

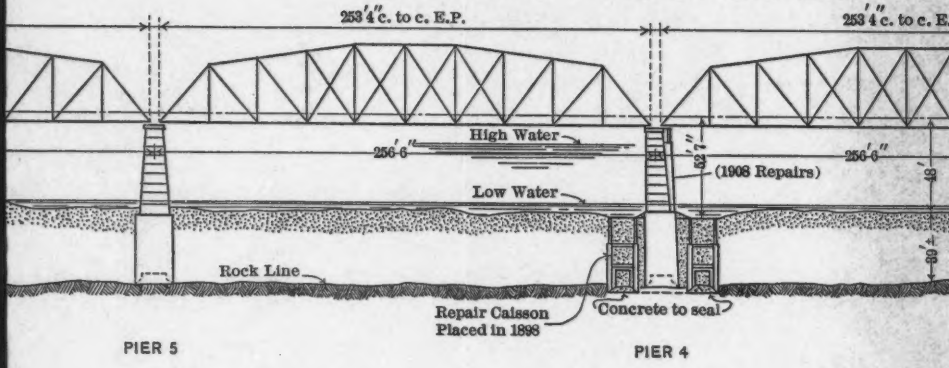


LITTLE ROCK JUNCTION BRIDGE

PLAN SHOWING CENTER LINE OF TRACK AFTER LEVELLING UP DRAW PIER AND PIER 4 IN



SIDE ELEVATION OF LITTLE ROCK JCT. BRIDGE



LITTLE ROCK JUNCTION BRIDGE

PLAN SHOWING CENTER LINE OF TRACK AFTER LEVELLING UP DRAW PIER AND F

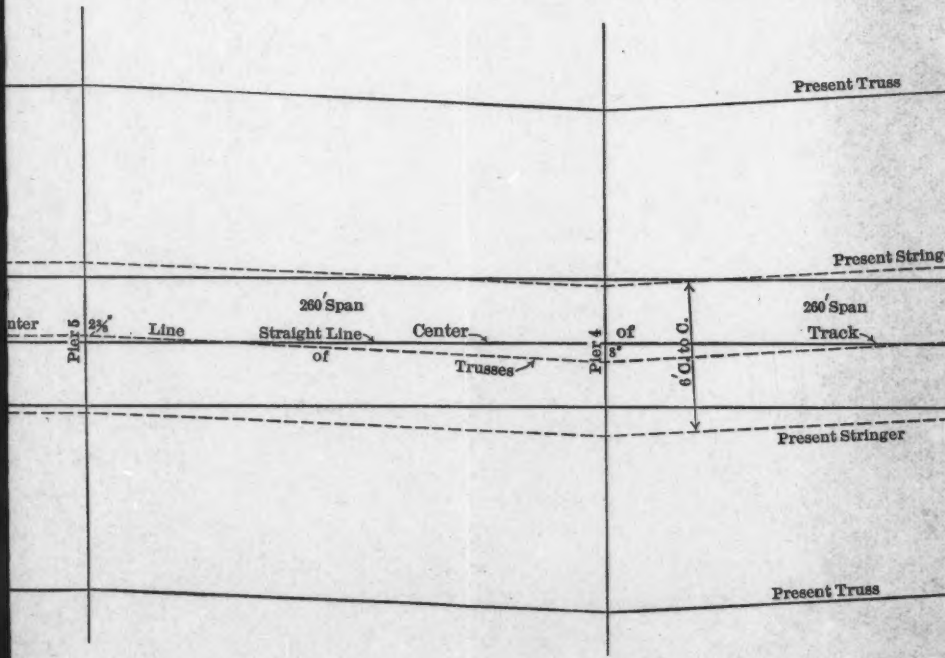
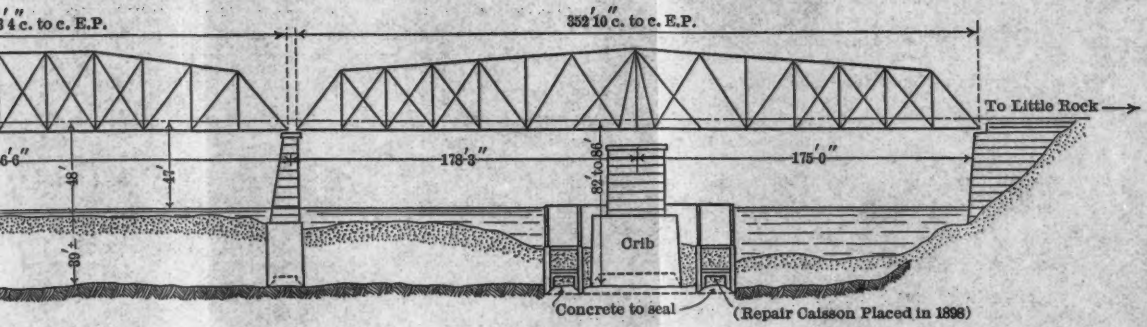
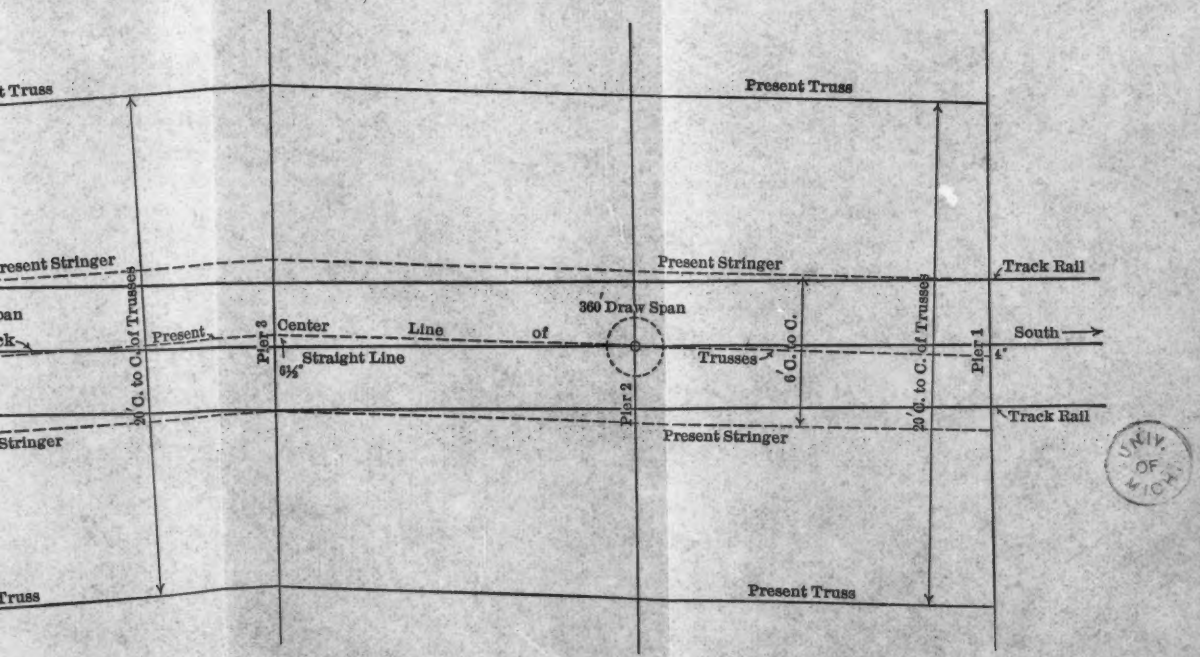


PLATE XXXII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1914.
SMITH ON
REPAIRING BRIDGE PIERS.



IER AND PIER 4 IN 1899



UNIV. OF MICH.

FIG. 2.—LITTLE ROCK JUNCTION BRIDGE, LITTLE ROCK, ARK.

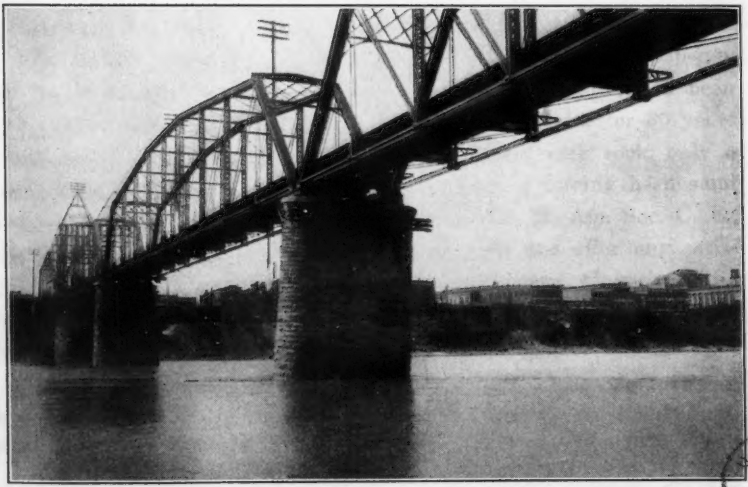


FIG. 3.—PIER 4 AFTER APPLICATION OF YOKES.

"Modifications may be made in these plans and specifications upon the request in writing by the General Manager of the Little Rock Junction Railway, and these modifications shall be complied with by the Contractors, but if the modifications increase the cost of the work, such increased cost shall be paid to the contractors."

The spans were so well designed and constructed that they are now safely carrying the present-day heavy engines and trains, and will continue to do so for many years. The design and construction of the south abutment and of Piers 6 and 7 were good and well executed. The design of the four pneumatic piers, although not in accordance with the best present-day practice, was adequate, but the construction was so faulty that trouble was experienced with them from the first, and the efforts that have been made during a period of 30 years to correct the defects resulted in a sequence of events that partook of the nature of a farce comedy in the face of impending disaster, which latter was narrowly averted.

There was nothing unusual or defective about the design or construction of the caissons, but they were very poorly located and carelessly controlled during sinking, resulting in their having been founded considerably out of place—from 2 to 3 ft. in one or two cases. The timber cribs, extending vertically upward from the caissons, reflected at their tops the improper location of the caissons.

The design evidently contemplated filling the cribs with rip-rap, and for 25 years it was not believed that any other filling had been used. After that time, however, it was suspected, and after 30 years it was learned, that, instead of having been filled with rock, only a small quantity had been used, most of the filling having been sand discharged from the caisson. C. de la C. Purdon, M. Am. Soc. C. E., who was inspector during the construction, does not offer any satisfactory explanation regarding the small percentage of rock filling and large percentage of sand filling, a letter from him in reference thereto reading as follows:

"The piers were built with ordinary caissons, and the caissons themselves filled with concrete. Above the caisson to about 4 ft. below the low-water mark, a crib was built. The timbers were either 3 ft. centers or 3 ft. clear—I don't remember exactly which—being all 12 by 12, and the spaces between these timbers, instead of being filled with concrete as is customary, were filled with rip-rap and then sand washed in. I objected to this strongly at the time, but as I

did not design the piers, this work being done by the late Mr. T. E. Sickels, I was not responsible for them. I told Mr. Wood at the time that I was satisfied the timbers would eventually crush and put the piers out of shape, which it seems occurred."

This does not entirely explain the trouble. Had the cribs been properly filled, the rip-rap filling every part, and then with sand washed into the interstices of the rip-rap, little, if any, settlement would have taken place. Very little rip-rap was placed, the greater part of the filling being sand, and, when this leaked out, the small quantity of rip-rap settled through the cribs, leaving the timbers to carry all the load. No satisfactory explanation has been found for filling the cribs with sand instead of stone.

The incorrect location of the caissons and cribs was discovered before starting the masonry, and as the spans were on the ground, the errors in location were corrected partly by placing the masonry piers to one side or the other of the cribs, partly in the batter of the piers, and partly in the placing of the bed-plates on the pier tops.

Pier 4 was built near the north edge of the crib and given equal batter on the two sides, and Pier 3, the north rest pier of the draw-span, was built near the center of the crib; but the north face was given a batter of 1½ in. per ft., and the south face was built plumb; the draw-span barely got a bearing on Pier 3, and the next fixed span reached well over on the pier.

Fig. 4 shows the location of the footing courses of Piers 3 and 4 with reference to the tops of the cribs, the cross-section of Pier 4, and the location of the intermediate span from Pier 3 to Pier 4.

Early Trouble.

The early record of the trouble is not clear, but it appears that, immediately after the completion of the bridge, the pivot pier under the draw-span (Pier 2) and Pier 4 began to settle and lean. As the bed of the Arkansas River is composed of fine sand which scours and shifts greatly during floods, it was thought that the settling was due to scour, the opinion immediately being formed that the cutting edges had not been founded on rock. Consequently, large quantities of rip-rap were unloaded around the piers, only to be washed down stream in following floods and requiring replacement. In addition, from time to time as necessity arose, the spans were

shifted back and forth, to keep their bearings on the piers, the tops of which had been made so small that very little variation could be permitted. The movement of the pivot pier was quite pronounced, and necessitated frequent leveling and adjustment of the draw-span, at great expense.

The matter was the subject of continual correspondence between the various officers, and, in reply to inquiry as to the best manner of strengthening the piers, Chief Engineer James W. Way, on December 6th, 1897, recommended as follows:

"Replying to your inquiry as to best method of proceeding with the work of strengthening the foundations of the pivot pier and pier 4 (numbering from the south) of the Little Rock Junction Bridge across the Arkansas River at Little Rock.

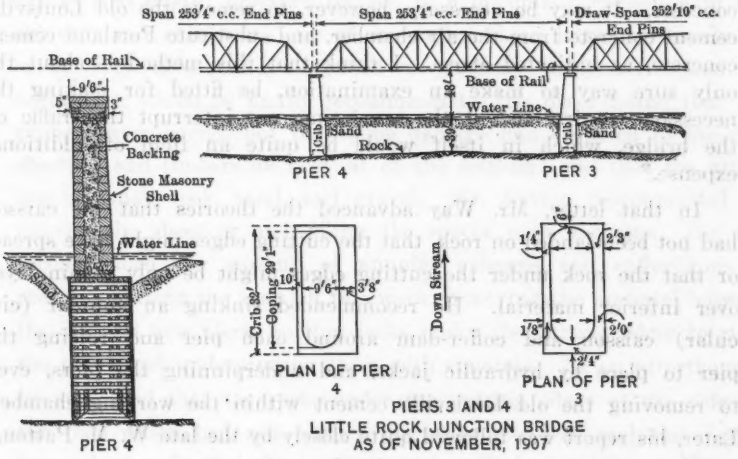


FIG. 4.

"One theory which may be presented for consideration is that the caissons are not properly landed on bed-rock, or possibly they may be faulty in construction and have spread at the cutting edges or on the corners. Again, it may be that what was termed bed-rock and believed to be such when the caissons were put in place, was but a thin stratum of shale overlying softer material; and that the respective weights of the piers, with their loads have caused this stratum to settle.

"This can readily be determined, if indeed it has not already been done, by making borings three or more feet below the elevation of the cutting edges of the caissons. Should the borings show solid

rock, then the natural inference is that the caissons are imperfectly constructed. I would therefore suggest that a careful examination of the bed of the river be made and the character of the obstructions ascertained. Boring should then be driven to disclose the nature of the rock on bottom. This should be done very carefully; and finally the caisson or crib should be inspected to ascertain condition of timbers, etc.

"A double steel caisson of sufficient diameter and with four or five compartments could then be sunk to bed-rock, using compressed air to remove the débris between the two cylinders. When this is done the space between the inside cylinder of the steel caisson and the wooden crib can be pumped and cleaned out to bed-rock; when an inspection will disclose the best method of correcting the trouble.

"It is possible that by the use of hydraulic jacks the pier can be raised to place and the air chamber filled with Portland cement concrete. It may be necessary, however, to remove the old Louisville cement concrete from the air chamber, and substitute Portland cement concrete, as indicated above. I think that this method is about the only sure way to make an examination, be fitted for making the necessary repairs, and at the same time not interrupt the traffic on the bridge, which in itself would be quite an item of additional expense."

In that letter, Mr. Way advanced the theories that the caisson had not been landed on rock, that the cutting edges might have spread, or that the rock under the cutting edges might be only a thin layer over inferior material. He recommended sinking an annular (circular) caisson and coffer-dam around each pier and raising the pier to place by hydraulic jacks, and underpinning the piers, even to removing the old Louisville cement within the working chamber. Later, his report was followed quite closely by the late W. M. Patton,* and large expenditures were made along those lines.

After the Railway Company's forces had handled the problem for 15 years, Mr. Patton was called in as Consulting Engineer. He studied the history of the bridge and, after making borings through the timber crib and caisson under the pivot pier, within the well inside the pier, and examining the exposed portions of the crib by divers, came to the conclusion that the caissons were not founded on rock, and gave the Railway Company a full and very plausible report and recommendation, which, on account of its unusual nature, is reprinted in full in Appendix A.

* Author of "Treatise on Foundations."

Patton's Report.

In brief, Mr. Patton concluded that one corner of the cutting edge of each pier rested on rock and the remainder on inferior material, which condition, together with the eccentricity of loading, caused the settlement; that greater scour at one corner than at the others caused the greatest settlement at the corner of greatest scour. He suggested pumping grout into the underlying sand to convert it into concrete; removing the inferior material under the caisson and replacing it with concrete, and, to accomplish the latter, discussed freezing the sand to the river bed to form a frozen sand coffer-dam. He also suggested an ordinary coffer-dam, which he dismissed on account of the difficulty of getting it tight in that material and at that depth; proposed the use of a pneumatic caisson with the old crib and caisson as one of its walls, which latter he concluded to be very risky; and suggested an entirely independent annular coffer-dam, which he finally recommended. His diver found that the timbers were sound, but that some damage had occurred to the crib sheeting and timbers on account of the settling; and that the filling was of small rock, sand, and gravel. Mr. Patton recommended replacing the sheeting and filling the spaces with concrete. He also recommended the sinking of annular caissons and coffer-dams to rock, supporting the span on falsework, removing the material between the old and new piers, putting jacks under the cutting edges to right the piers, and underpinning them with concrete. As an afterthought, he suggested that the rock, under the cutting edges at the sides of the pier which had not settled, might be cut out, and those edges lowered to make the piers assume perpendicular positions. He went so far as to say that from 30 to 35 hydraulic jacks, with main cylinders $3\frac{1}{2}$ in. in diameter and plungers $\frac{3}{4}$ in. in diameter, would raise 30 tons each. It is not known how he proposed to work the jacks when under the cutting edge, as he recommended letting the annular caisson flood while jacking so as to reduce the weight to be lifted. His estimated cost was $40 000 for the coffer-dam around the pivot pier, with $4 000 for raising and leveling it, and $34 000 and $3 500, respectively, for Pier 4, a total of $81 500.

It is puzzling that Mr. Patton did not determine by simple calculation that, if Pier 4 could be righted by such a method as he suggested,

the top would be in such a position that the ends of the spans supported by it would not fit the top of the pier in the new location, as it would have been moved over 3 or 4 ft. It also appears that he ignored the statements of the contractor and resident engineer who built the piers that they were surely founded on rock. Later, the sinking of the coffer-dam around Pier 4 disclosed the fact that the rock slopes in a direction opposite to the slope on which he based his theory, a fact that could easily have been determined by running jet pipes down to rock at several places around the pier.

ANNULAR COFFER-DAMS SUNK IN 1898 AND 1899.

Mr. Patton's report was accepted, and bids were requested on his plans and specifications, with the following results:

Engineering-Contracting Company. (Charles Sooysmith, M. Am. Soc. C. E.):

Pivot pier, $38 332. $1 500 per ft. below 40 ft. below low water.

Pier 4, $36 010. $1 200 per ft. below 45 ft. below low water.

$18 per cu. yd. for work below the cutting edge of both piers.

Missouri Valley Bridge and Iron Company. (The late A. J. Tullock, M. Am. Soc. C. E.):

Pivot pier, $31 600. $800 per ft. below 40 ft.

Pier 4, $27 950. $750 per ft. below 45 ft.

Removal of material between new coffer-dam and old pier and all other work, cost plus 10 per cent.

McGee-Kahmann and Company, Kansas City:

Both piers, $116 000.

Excavation of material between, $1 per cu. yd.

Three companies made propositions for building new piers, as follows:

McGee-Kahmann and Company proposed to build four new piers, 50 ft. north of the present piers, launch a draw-span 50 ft. north, and construct a new span 50 ft. shorter between Piers 5 and 6 and a deck plate-girder at the south end, for $99 000. They also made an alternate bid to build a new draw-span, 80 ft. longer than the old one, reaching from the south abutment to the new piers, renewing Piers 2, 3, and 4, farther north, and to build a new span between Piers 4 and 5, for $125 000. The latter plan would have left in the defective Pier 5.

The Missouri Valley Bridge and Iron Company proposed to build an entirely new set of piers according to the best modern practice, for $125 000.

All propositions for new piers were rejected, and the bid of the Missouri Valley Bridge and Iron Company for going ahead on Mr. Patton's recommendation was accepted. Mr. A. J. Tullock, then Proprietor of the Missouri Valley Bridge and Iron Company, had made a careful investigation before bidding; the correspondence accompanying his bid pointed out the weaknesses of Mr. Patton's scheme, and made certain recommendations. On account of the value of Mr. Tullock's conclusions, his letters of August 17th, 18th, and 19th, 1898, to Mr. E. Fisher, then Engineer of Bridges and Buildings, of the Railway Company, are given in full in Appendix B.

As a result of his study, Mr. Tullock raised a doubt as to the correctness of the theories formerly advanced regarding the trouble, and attributed it to the crushing of the crib timbers. He pointed out the impracticability and probable impossibility of carrying out the Patton scheme, and recommended against it, giving as his opinion that it was too dangerous to warrant its adoption. He recommended that the cribs be cleaned out and filled with concrete. Although recommending against the repair work, he submitted the lowest bid for it, and received the award.

The work of sinking the annular caissons and coffer-dams went forward in the fall of 1898, and was completed early in the summer of 1899.

At Pier 4, a timber caisson, about 50 by 70 ft., was constructed around the old pier, the intention being to carry out the work, as nearly as practicable, in accordance with Mr. Tullock's proposal of August 17th, 1898 (Case 2, Appendix B).

Two rows of piling were driven around the old pier, about 8 or 10 ft. apart, and were capped and cross-capped; the caisson and crib were built on top of these caps to a height of 12 ft. Large screws, 16 ft. long, were then used to lower the crib until it floated. The crib projected 4 ft. above the water, and the cutting edge floated about 4 ft. above the bed of the river; sand was used to fill the crib to sink it, while men were raising timber on top of the crib.

On March 1st, 1899, with the cutting edge on the bed of the river at an elevation of 11 ft. below zero, air was forced into the working chamber and excavation was made through the following material:

From Elevation 11 to 17.....Sand.
From Elevation 17 to 23.....Rip-rap and sand.
From Elevation 23 to 41.....Sand.

On April 2d, 1899, at 41 ft. below low water, hard shale was struck near the center of the up-stream end. From 41 to 45 ft. below zero, excavation was made through very hard black shale on the north side and at the up- and down-stream ends. This rock was nearly as hard as granite, requiring dynamite to make the excavation.

At an elevation of 45 ft., the inside cutting edge on the south side was landed on rock, except about 10 ft., where the rock was 6 in. below the edge. The rock under the outside cutting edge on the south side was from 12 to 13 in. below it. However, the sand was taken out down to the rock, and the working chamber was sealed with from 2 to 4 ft. of concrete, depending on the depth of rock.

The crib was built 58 ft. high, the top being 13 ft. above low water when the caisson was sealed. Later, the top of the crib was disconnected, from 6 to 7 ft. below low water, after concrete had been placed between the old and new cribs. The sealing of the working chamber was completed on April 24th, 1899.

The river began rising very rapidly on April 22d, and on the 23d a 7-ft. rise was reported at Fort Smith. This reached Little Rock a few hours after the work of sealing the working chamber had been completed, the water rising over the top of the crib. Work was suspended from April 25th until June 30th, and then two pumps and an air-lift were used to remove the water between the old and new cribs, and men commenced excavating the loose sand and sediment from around the old pier, but no effort was made to go below the depth uncovered by the pumps, or to remove the silt. The silt and sand were cleaned out to a depth of 5 ft. on the up-stream and 7 ft. on the down-stream end below the top of the old crib. Investigation of the timber in the old crib disclosed crushing on the north side, but only to the extent of compressing the timber, and not dividing the grain. The timber was good and sound.

A great deal of the 3-in. sheeting was found to be torn off, and it was seen that the spaces between the timbers of the crib were filled with sand and loose rock. The sheeting which had been torn off was replaced. It was also observed that the deck or top of the old crib dipped from 1 in. to 1½ in. from a point near the center on the south side of the pier toward the up-stream end.

On July 1st, while the loose material between the new coffer-dam and the old crib was being excavated, it was discovered that the pier had settled about ½ in. This led to the immediate abandonment of the excavation, but the spaces on the north and west sides were driven full of piles, with a penetration of about 15 ft., and the space already excavated was filled with concrete up to the top of the old crib. None of the material within the old crib was removed, and no effort was made to grout it or to fill in the vacant spaces with concrete. In fact, the sheeting that was replaced effectually prevented the concrete from flowing into the vacant spaces within the crib. The concreting around the old pier was completed on July 6th, 1899, and both walls of the annular coffer-dam were then removed, down to 6 or 7 ft. below low water.

The location of the new annular caisson and coffer-dam around Pier 4, with reference to the old crib, together with the general construction of both, is shown by Fig. 5, which also shows in cross-section the concrete, from 3 to 5 ft. thick, that was placed between the old crib and the new coffer-dam. The annular caisson was about 50 ft. wide and 70 ft. long, the sides were 10 ft. apart, and spaces from 6 ft. to 7 ft. 6 in. wide were left between the old and new work.

The work at the pivot pier was similar in character, except that the new caisson was 70 ft. square and the concrete about 15 ft. thick, the extra depth having been caused by scour and not by excavation. (Fig. 5 does not show correctly the pier top at this time (1898), but as it appeared several years later, after additional movement had taken place.)

Mr. Tullock considered the removal of the material between the new and old caissons to be very hazardous, and he continued to impress this opinion on the officials of the Railway Company. He was so much impressed by it that he continually recommended and urged that only part of the material be removed and that the remainder be confined by placing concrete over it. Unfortunately, the Railway Com-

pany was guided by his advice in this matter. Two of Mr. Tullock's letters to Mr. Fisher, relative to this matter, dated November 24th, 1898, and May 16th, 1899, are printed in full in Appendix C.

It is to be regretted that, after having incurred the great expense of sinking the annular caissons, more benefit was not derived from them. The scheme of jacking up the piers and underpinning them

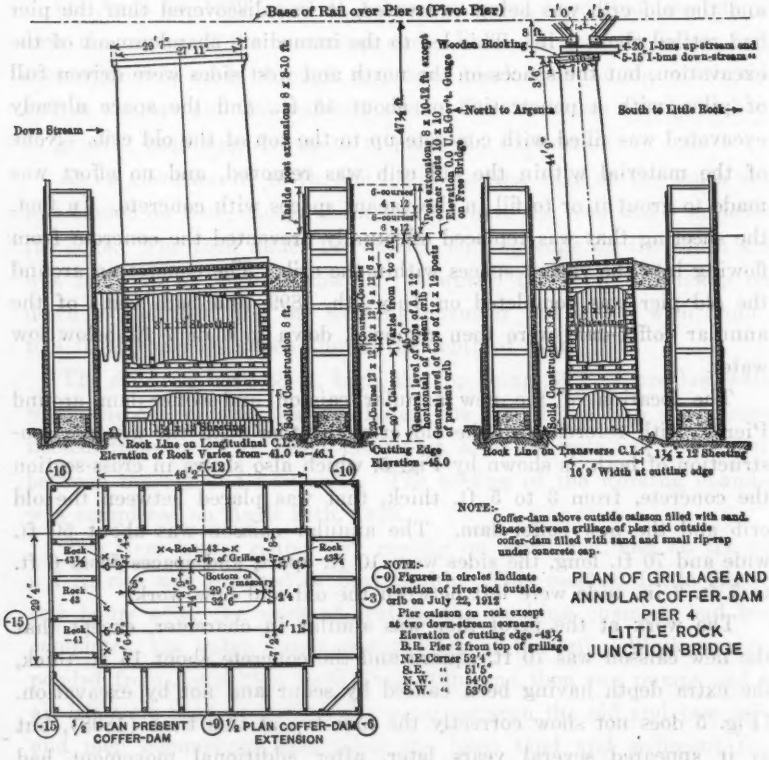


FIG. 5.

appears to have been abandoned early, as no reference to these matters can be found in the files during the work. Neither was any effort made, by exploring under the cutting edges of the new caissons, to determine the depth to which the original caissons had been sunk.

During May, 1899, the concrete having been placed around the pivot pier, the wheel-tread under the drum of the draw-span was leveled up

and underpinned with concrete, the minimum rise being 2⅜ in. and the maximum 6½ in.

At the same time, but prior to the placing of the concrete around Pier 4, the spans on that pier were also leveled up and carried on oak blocking in place of the bed-plates, the spans being raised from 1¼ to 6 in. This work was done too soon, as a slight settlement of Pier 4 during the work threw the track out of line.

The pier tops had moved around so much, and their upper surfaces were so small, that the trusses could not be placed on a straight line. The trusses had been built with extra wide clearance to provide for highway traffic, and this permitted considerable latitude in the location of the track. The trusses were permitted to remain out of line, and the track was placed on a tangent, being nearer to one truss than to the other where the spans were out of line. The location of the center line of the track with reference to the track stringers and trusses is shown on Plate XXXII.

Further Movement of Pier 4.

On account of the large quantity of concrete placed around the pivot pier, and the depth to which it reached, 15 ft. below the top of the crib, no further movement has been detected in this pier. However, the movement of Pier 4 was not arrested, but as the settlement and movement continued, a policy of "watchful waiting" was followed for several years. The situation again became critical early in 1906, when Pier 4 had moved so far that one of the spans was in imminent danger of falling off.

At that time J. C. Bland, M. Am. Soc. C. E., was retained as Consulting Bridge Engineer of the Missouri Pacific Railway, and was asked to look into the matter and pass on it. He arranged to have John N. Ostrom, M. Am. Soc. C. E., to visit the bridge and report on it. That portion of Mr. Ostrom's report having reference to the foundations is as follows:

"The roller nests at north end of span, which is next to the draw-span on the north, are badly shifted to the south, so that two of the rollers do not bear.

"The main trouble with the bridge is in the foundations, and, like the Baring Cross Bridge, about one mile above, there are no original records showing the exact conditions of the bearings on the bottom.

The pivot pier, Pier 3 and Pier 4, on the north side of the pivot, are all tilted up stream and also sideways.

"The pivot pier and Pier 4 have been protected by concrete additions extending clear around the base on the outside and reaching down into the rock. It is believed that the tilting of the pivot pier, at least, has been arrested.

"The piers also have well-defined vertical cracks extending through several courses, indicating uneven bearing on the bottom, but the cracks are not bad enough to endanger the structure at present.

"The south abutment has a deep crack in the up-stream wing-wall, and the north face next to the river has several bands, one above the other, held by bolts which pass south into the abutment.

"Pier 7 on the north shore is cracked, and banded with tie-rods running through yokes on the up- and down-stream ends.

"The most important matter at present is to make careful measurements at least once a month on all the piers and abutments with a view of determining whether or not the tilting and cracking has stopped. If these measurements show that the masonry has become fixed in position, it may be assumed safe for the life of the present superstructure, but it would not be suitable for a new bridge designed under present specifications.

"If the cracks in piers increase, they should be banded, and the cracks should be pointed as soon as practicable, to exclude water.

"Owing to the tilting of foundations, the surface and alignment of the track is very bad on the bridge. As the piers have tilted up stream the track has been shifted down stream to keep the alignment as good as possible until the down-stream stringers are carrying considerable more than half the load. Furthermore, the clearance between the train and the down-stream truss has been reduced by the amount of shifting of the track."

"JNO. N. OSTROM."

"To MR. J. C. BLAND.

"MAY 21, 1906."

In transmitting Mr. Ostrom's report to M. L. Byers, M. Am. Soc. C. E., then Chief Engineer of Maintenance of Way, Mr. Bland wrote in part as follows:

"JUNE 4, 1906.

"M. L. BYERS.

"One can hardly tell much about the masonry tilting. The plans you sent show the tilting but do not say the character of foundation, nor whether the condition is becoming worse. You will notice that Mr. Ostrom says:

"'It is believed that the tilting of the pivot pier, at least, has been arrested.'

"Who is it that says so, and on what grounds?

"I think you should use slow speed over the bridge, and also I would prohibit the stopping and starting of trains on bridge, if such an order is practicable.

"J. C. BLAND."

Following this report no immediate action was taken.

One of the first duties assigned to the writer when he entered the Bridge Department of the Railway Company, late in 1907, was an inspection of this bridge in order to determine how to maintain it in safe condition. At that time, the location of the pedestals of the spans with reference to the coping courses on Piers 3, 4, 5, and 6, was as shown in Fig. 6. Pier 4 had moved out so far from under the shoes that the center of the end pins was almost exactly over the edge of the timber blocking under the shoe; the edge of the shoe overhung the edge of the pier, as shown in Fig. 7.

PLAN SHOWING RELATIVE POSITIONS OF
TRUSS PEDESTALS AND TOP OF PIERS
LITTLE ROCK JUNCTION BRIDGE
DEC. 1907

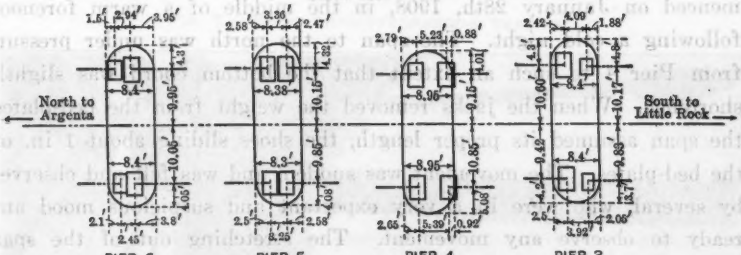


FIG. 6.

The roller end of the span to the south being on Pier 4 permitted the pier to move out from under. The fixed end of the span to the north being on Pier 4, the thrust caused by the leaning of the pier in that direction had crowded the roller end of this span on Pier 5 hard against the fixed end of the next span. This thrust, added to the similar, though less, defective construction of Pier 5, caused the latter to lean north, crowding the roller end of the next span to the north, hard against the fixed end of the north span on Pier 6, bringing three spans in hard contact. On hot days, when the bridge was not loaded, the chord bars in the end panels of these spans could be seen to be more or less buckled, but they always straightened out

under trains, undoubtedly because the elasticity of the timber cribs permitted the piers to move back and forth.

Immediately following his first visit to the bridge, the writer worked up plans for placing a nest of I-beams under the truss shoes at each end of Pier 4, so that the overhanging ends of these beams would afford support for the end of the next span south to keep that span in the air instead of in the water. Fig. 9 shows the arrangement of I-beams for this purpose. There was only sufficient depth for 15-in. I-beams at the high end of the pier, but at the low end, there was sufficient depth for 20-in. beams, the difference in height being due to the settlement.

The plans first contemplated driving piles on both sides of Pier 4 to be used for jacking during the placing of the I-beams, but, on account of the extreme length of the piles required (from 80 to 90 ft.) and the hazard attending their maintenance in the Arkansas River, it was decided to avoid their use and jack from the pier top.

The jacking arrangement was set in place and jacking was commenced on January 28th, 1908, in the middle of a warm forenoon following a cold night. The span to the north was under pressure from Pier 4 to such an extent that the bottom chord was slightly shortened. When the jacks removed the weight from the bed-plates, the span assumed its proper length, the shoes sliding about 1 in. on the bed-plates. The movement was sudden, and was felt and observed by several, who were in a very expectant and suspicious mood and ready to observe any movement. The stretching out of the span was overlooked, as it was not noticed immediately that no corresponding movement had taken place under the roller end of the other span on the same pier. The pier had such a bad record that all the men engaged on the work believed it had suddenly moved and was going to fall, and the forces, led by the foreman, fled from the structure, headquarters being advised by telegram as follows:

"Began jacking bridge this A. M.; raised 1 in. when Pier 4 moved north 1 in. Will be necessary to yoke Piers 3 and 4 to span before it will be safe to handle driver for falsework. It will be necessary to put spans on falsework before can resume traffic."

As an emergency matter, to retard the movement of Pier 4 and to avoid any such scare during the work, the yokes and rods shown

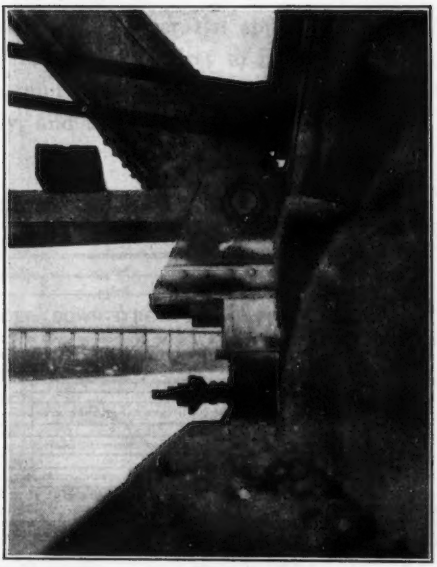


FIG. 7.—TRUSS SHOE OVERHANGING BEARING.

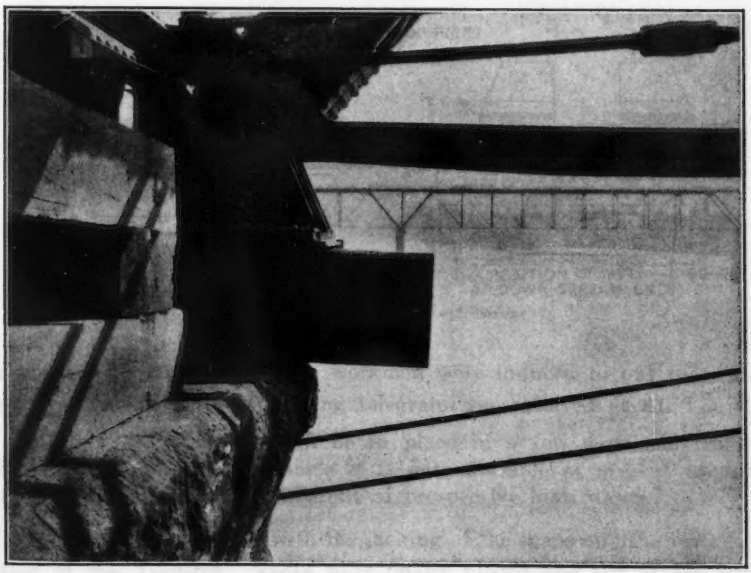


FIG. 8.—TRUSS SHOE ON I-BEAM GRILLAGE.

by Fig. 10 had been designed, but the conclusion was reached that they would not be necessary. After the apparent movement of the pier, however, it was impossible at first to induce the workmen to return to the structure, and arrangements were completed to make the rods, etc., immediately, and to apply them—for their moral effect as much

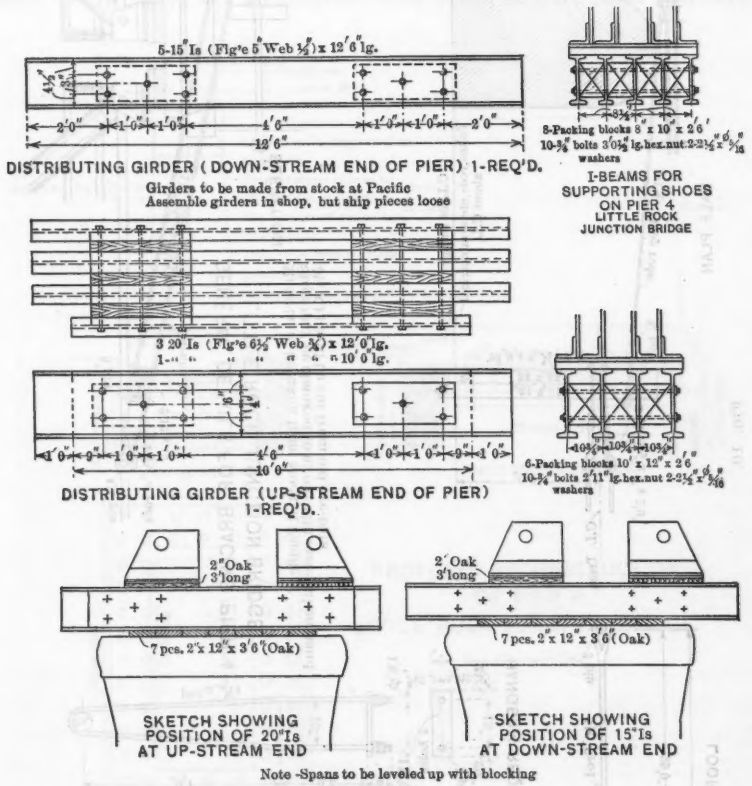


FIG. 9.

as for any other reason. The workmen were induced to put them in place, after which the following telegram was received at St. Louis:

"Yokes and I-beams will be in place in a few days and traffic then can be resumed. Believe it is not advisable at present to put falsework under spans on account of prospective high water."

They then went ahead with the jacking of the spans and the installation of the I-beams. These were finally placed and the bridge was

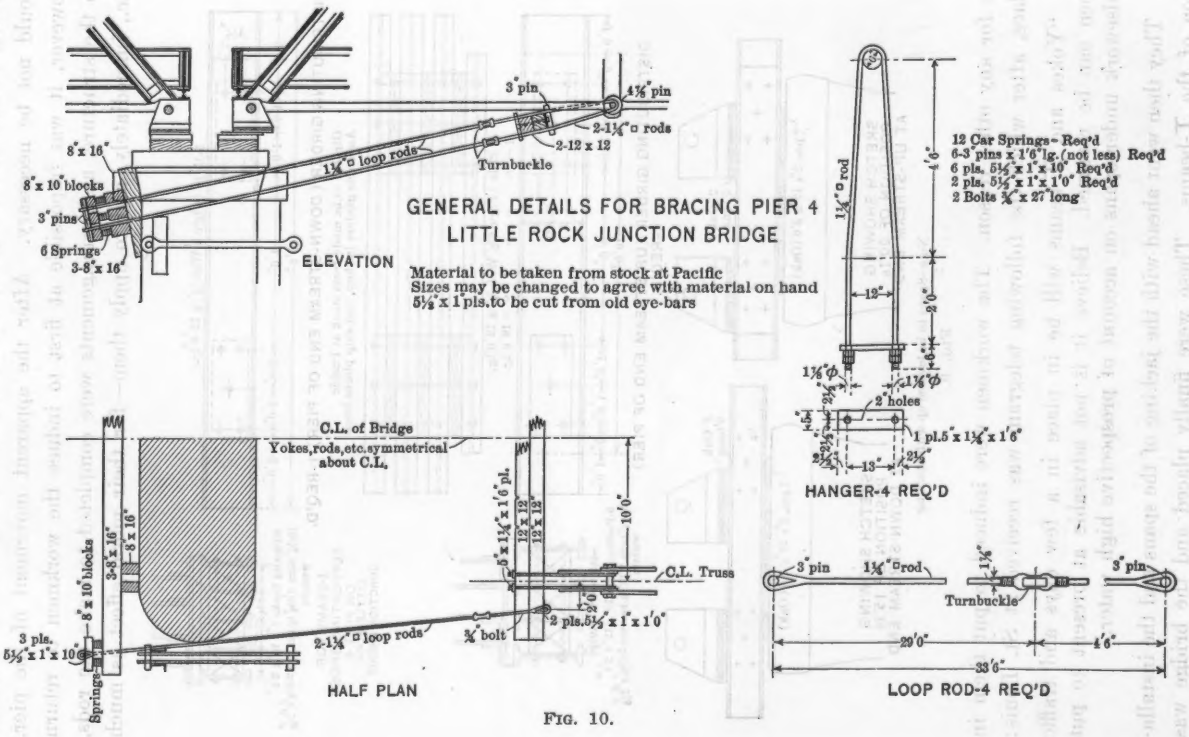

3" pin
1⅛" pin
2-1¼"□ rods
2-12 x 12
Turnbuckle
8" x 16"
8" x 10" blocks
1¼"□ loop rods
3" pins
6 Springs
3-8" x 16"
ELEVATION
GENERAL DETAILS FOR BRACING PIER 4
LITTLE ROCK JUNCTION BRIDGE
Material to be taken from stock at Pacific
Sizes may be changed to agree with material on hand
5½" x 1" pls. to be cut from old eye-bars
12 Car Springs - Req'd
6-3" pins x 1'6" lg. (not less) Req'd
6 pls. 5½" x 1" x 10" Req'd
2 pls. 5½" x 1" x 1'0" Req'd
2 Bolts ¾" x 27" long
1¼"□ rod
1'6"
12"
2'0"
1⅛"Φ
2" holes
1 pl. 5" x 1¼" x 1'6"
13"
HANGER-4 REQ'D
C.L. of Bridge
Yokes, rods, etc. symmetrical about C.L.
5" x 1¼" x 1'6" pl.
12" x 12"
10'0"
C.L. Truss
2 pls. 5½" x 1" x 1'0"
¾" bolt
2-1¼"□ loop rods
3 pls. 5½" x 1" x 10"
Springs
HALF PLAN
3" pin
1¼"□ rod
Turnbuckle
29'0"
4'6"
33'6"
LOOP ROD-4 REQ'D

FIG. 10.

restored to service on February 1st, 1908, 4 days after the commencement of the jacking.

After placing the I-beams, the spans were shifted slightly to remove their interference, and the rollers were cleaned and oiled. The yokes were left in place, and instructions were issued to have them tightened

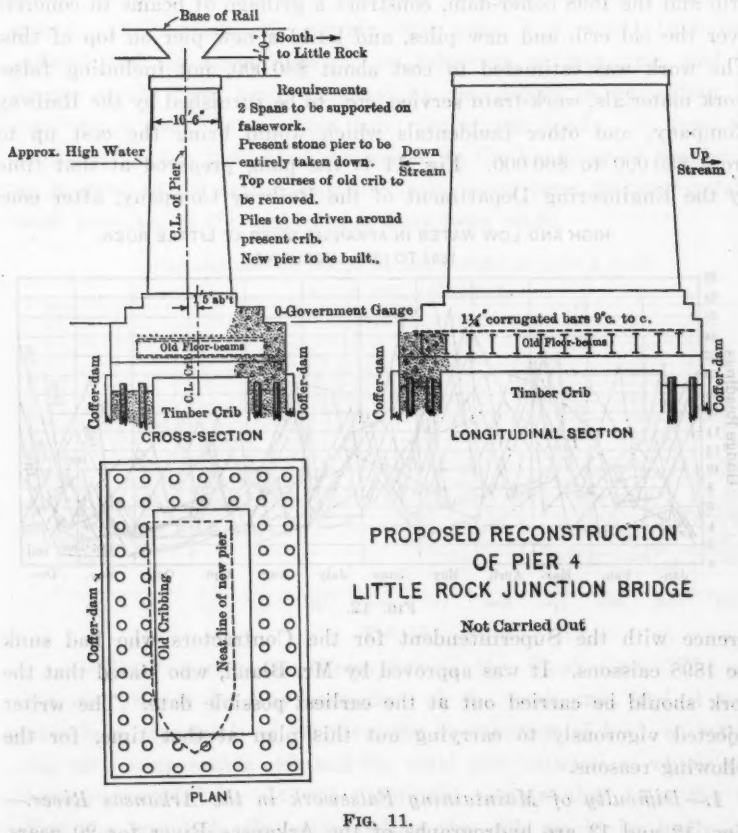


FIG. 11.

every month. No further attention was paid to the placing of the falsework. The condition of the bearing under the expansion end of the span to the south, before the I-beams were placed, is shown by Fig. 7. Fig. 8 shows the condition afterward. The bearing having been made safe, the matter of permanent repairs was then given further study.

Plans for Reconstruction of Pier 4.

Late in 1907 tentative plans had been made to replace Pier 4 by a larger one. It was planned to place the two spans resting on Pier 4 on falsework, take down the masonry pier to the top of the crib, drive as many piles as possible in the annular space between the old crib and the 1898 coffer-dam, construct a grillage of beams in concrete over the old crib and new piles, and build a new pier on top of this. The work was estimated to cost about $40 000, not including falsework materials, work-train service, etc., to be furnished by the Railway Company, and other incidentals which would bring the cost up to from $50 000 to $60 000. Fig. 11 is the plan, prepared at that time by the Engineering Department of the Railway Company, after con-

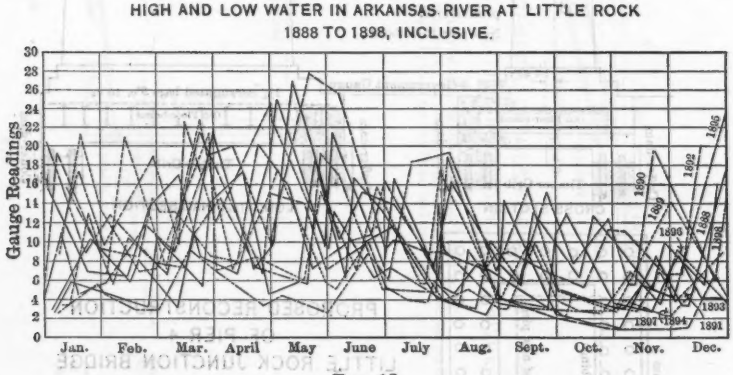


Fig. 12.

ference with the Superintendent for the Contractors who had sunk the 1898 caissons. It was approved by Mr. Bland, who stated that the work should be carried out at the earliest possible date. The writer objected vigorously to carrying out this plan at that time, for the following reasons.

1.—Difficulty of Maintaining Falsework in the Arkansas River.—Figs. 12 and 13 are hydrographs of the Arkansas River for 20 years, 1888 to 1908, and show that in the spring, when it would have been necessary to maintain two 253 ft. 4-in. spans on falsework with 80-ft. piles in the middle of the river, the floods are greatest. They also show that sudden rises can be expected in any month of the year, and are greatest in spring and least in early fall. Rises in the Arkansas are invariably rapid, and are characterized by excessive

quantities of drift, some of it whole trees, 80 or 90 ft. long, from 2 to 3 ft. in diameter and full of branches, and also by deep and rapid scour, down to rock, of the fine sand and silt forming the bed, especially around obstructions of any nature. In the past the Arkansas has taken heavy toll in falsework and spans, and there is hardly a bridge across the river where there has not been some trouble from this source. It is practically certain that great loss would have been suffered had the spans been put on falsework at that time, and the danger would have been increased by the delays in rebuilding the pier, which could be expected from the spring floods. In fact, this consideration has influenced the writer, in the handling of this bridge up to date, to the extent that piles have never been driven for falsework purposes in the repairs that have been made.

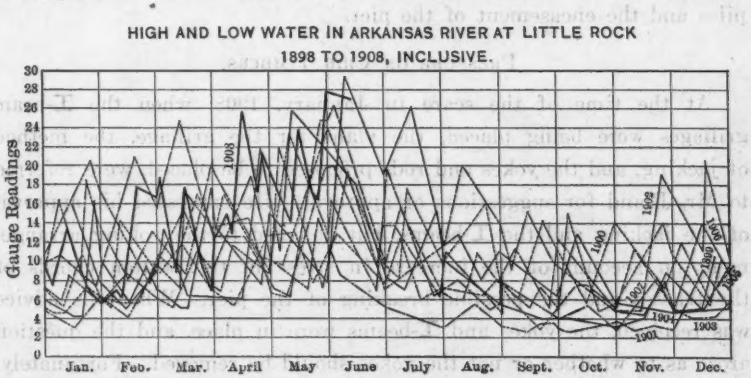


FIG. 13.

2.—No Provision for Strengthening the Weak Crib.—The plan did not provide for any correction of the weak crib, and it would have been impossible to calculate the distribution of load, as between the very compressible crib and the rigid pile foundation, which would have surrounded it. It was practically certain that from the very first the proposed piles surrounding the crib would be badly overloaded, and failure of some sort could be expected. In fact, the proposed pier was so much larger than the old pier that, if the load could have been distributed uniformly over the base, the weak crib would have been required to carry more load than before.

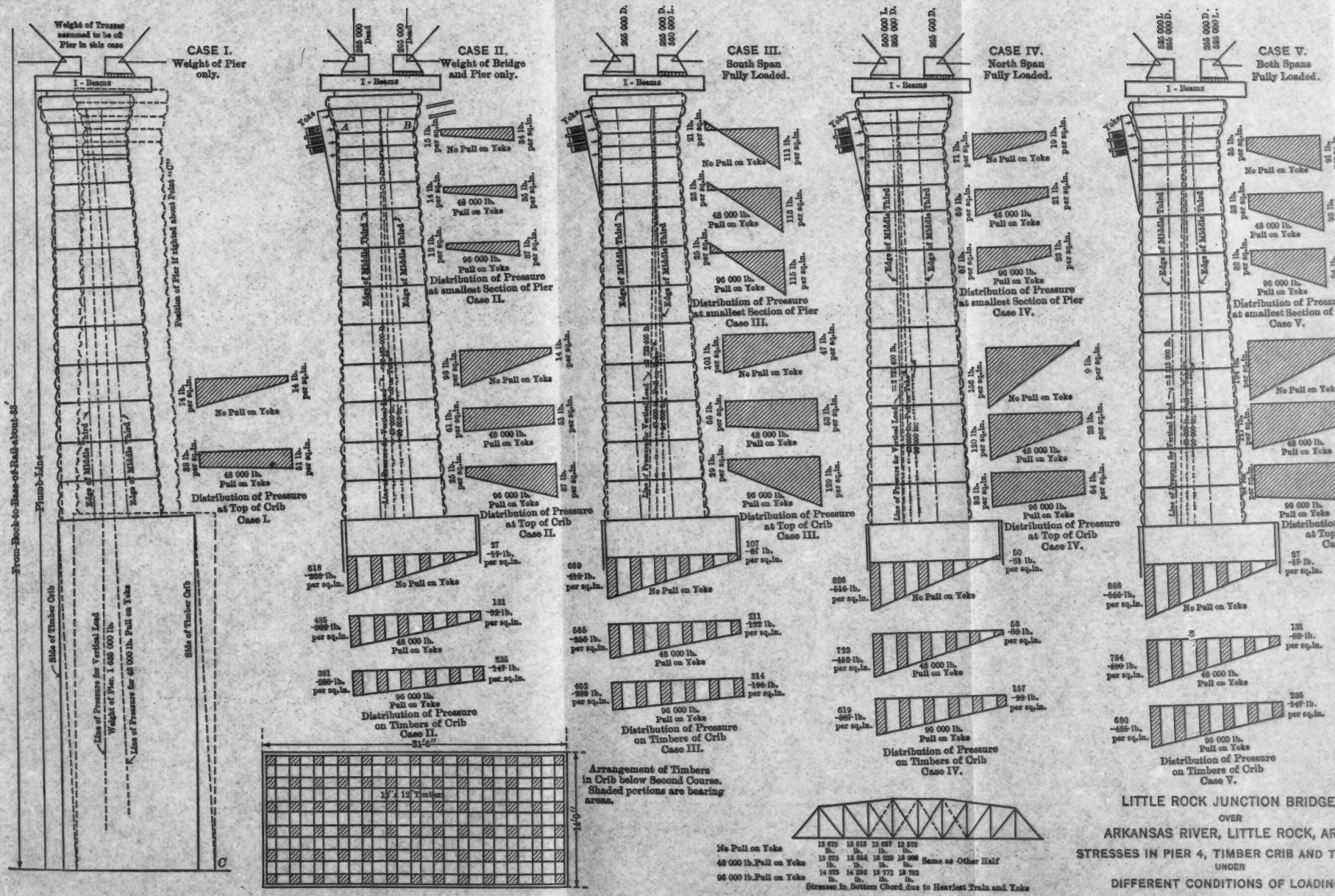
3.—Better results could have been obtained at greatly reduced expense by driving piles as proposed and building up on them a

reinforced concrete shell enclosing the old pier, thereby saving the latter and avoiding the cost of its renewal, all the cost and hazard of falsework, and, to a great extent, the uncertainty as to the distribution of the loading. The art is sufficiently advanced to insure absolutely efficient bond between the new encasement and the old masonry (the old was rough rock-face). The reduced uncertainty as to the distribution of the load is due to the fact that from the first the crib would carry its maximum load and be fully compressed during the construction of the encasement, so that the new support would work well with the old.

These three reasons were sufficiently potent to prevent the adoption of this scheme, but it was not possible for the writer to secure in the near future authority to go ahead with the driving of additional piles and the encasement of the pier.

Pressure on Crib Timbers.

At the time of the scare in January, 1908, when the I-beam grillages were being placed, the plans for the grillage, the method of jacking, and the yokes and rods proposed to be placed, were referred to Mr. Bland for suggestions or approval. He expressed his approval of the jacking and the I-beams, but objected to the yoking arrangement on account of the increase in stress in the bottom chords of the trusses and the possible breaking of the pier. When his advice was received, the yokes and I-beams were in place, and the question arose as to whether or not the yokes should be removed. Fortunately, no one was prepared immediately to assume the responsibility of removing them, and as the writer had suggested putting them on, he was requested to show cause why they should be permitted to remain. Rough figures had previously been prepared, showing the pressure on the crib timbers and the effect of the yokes, but to show better the beneficial effect of even a moderate pull near the top of the pier, the lines and intensities of pressure were determined for all probable conditions of loading and assumed stresses in the yokes. Plate XXXIII shows the results, and indicates that even a moderate pull at the top of the pier would be of considerable advantage. The nuts were tightened from time to time, and the springs were kept at such a pressure as to maintain a pull of about 50 000 lb. As the end panels of the bottom chords of the trusses had 48 sq. in.



LITTLE ROCK JUNCTION BRIDGE
OVER
ARKANSAS RIVER, LITTLE ROCK, ARK.
STRESSES IN PIER 4, TIMBER CRIB AND TRUS
UNDER
DIFFERENT CONDITIONS OF LOADING

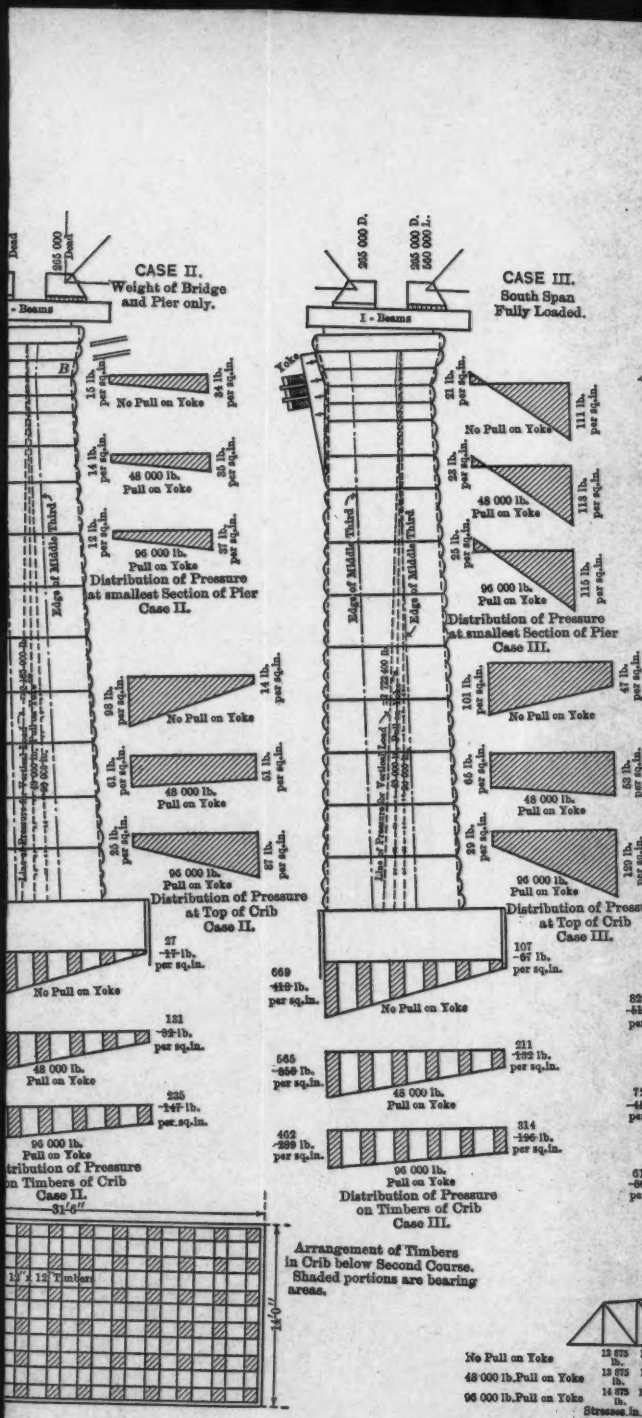
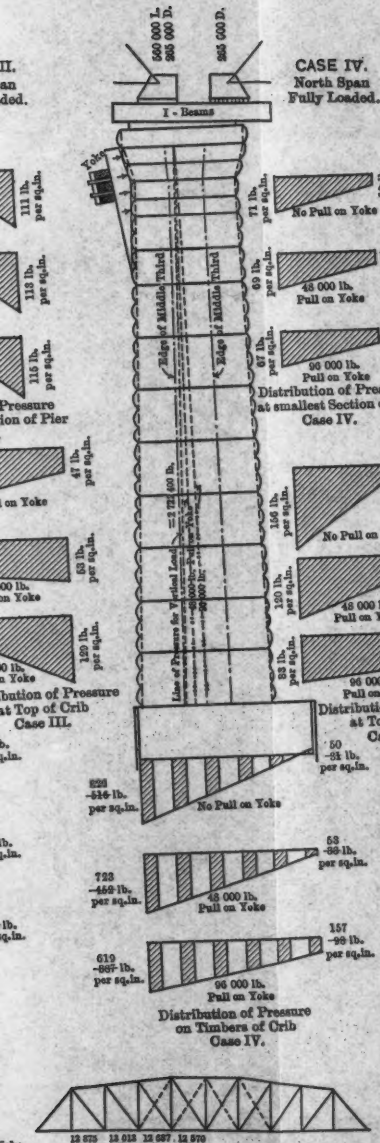

CASE II.
Weight of Bridge and Pier only.
Beams
B
No Pull on Yoke
48 000 lb. Pull on Yoke
96 000 lb. Pull on Yoke
Distribution of Pressure at smallest Section of Pier Case II.
Distribution of Pressure at Top of Crib Case II.
No Pull on Yoke
48 000 lb. Pull on Yoke
96 000 lb. Pull on Yoke
Distribution of Pressure on Timbers of Crib Case II.
CASE III.
South Span Fully Loaded.
I - Beams
Yoke
No Pull on Yoke
48 000 lb. Pull on Yoke
96 000 lb. Pull on Yoke
Distribution of Pressure at smallest Section of Pier Case III.
Distribution of Pressure at Top of Crib Case III.
No Pull on Yoke
48 000 lb. Pull on Yoke
96 000 lb. Pull on Yoke
Distribution of Pressure on Timbers of Crib Case III.
Arrangement of Timbers in Crib below Second Course. Shaded portions are bearing areas.
12" x 12" Timbers
No Pull on Yoke
48 000 lb. Pull on Yoke
96 000 lb. Pull on Yoke
Stresses in

PLATE XXXIII.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1914.
SMITH ON
REPAIRING BRIDGE PIERS.



Stresses in Bottom Chord due to Heaviest Train and Yoke

Yoke	12 575 lb.	13 013 lb.	12 687 lb.	12 870 lb.	
Yoke	12 575 lb.	13 053 lb.	13 229 lb.	13 098 lb.	Same as Other Half
Yoke	14 575 lb.	14 295 lb.	13 771 lb.	13 782 lb.	

LITTLE ROCK JUNCTION BRIDGE

OVER

ARKANSAS RIVER, LITTLE ROCK, ARK.

STRESSES IN PIER 4, TIMBER CRIB AND TRUSSES

UNDER

DIFFERENT CONDITIONS OF LOADING

UNIV. OF MICH.

of cross-section, figures of 48 000 and 96 000 lb. were assumed in the calculations, for the amounts of pull corresponding to 1 000 and 2 000 lb. per sq. in., respectively, the latter figure having also been used in the calculations to ascertain the results if the pull should reach double the amount assumed. The results showed conclusively that the trusses and the masonry pier were not adversely affected by the pull of the rods. The maximum stress in the bottom chord under the heaviest loading was about 13 000 lb. per sq. in. before the application of the rods and about 15 000 lb. per sq. in. afterward, an amount well within safe limits.

In order to avoid damage to the pier by the concentration of the pull of the yokes, bearing timbers had been cut to fit the shape of the rock face of the pier and placed in contact with the pier for a height of about 10 ft., resulting in spreading the greater part of the pull over four or five courses.

Fig. 10 shows the yokes and rods. The arrangement consisted of three 8 by 16-in. timbers laid horizontally and blocked against the side of the pier toward which it was leaning, the ends projecting beyond the pier ends and being attached at each end to two 1¼-in. round rods with upset ends. The rods passed close by the pier and extended up to the first panel points of the trusses, where, by the timbers and U-rods, the pull was transmitted to the bottom chord of the trusses. Heavy car springs were introduced at points of bearing partly to take up the expansion and contraction. The other end of the span to which the yoke had been connected was attached to Pier 3 by a duplicate of this arrangement.

The line of pressure is well within the middle-third for all cases except when the south span only is fully loaded, under which condition the line is outside the middle-third for the top 20 ft. The compression on one side and the tension on the other side did not reach intensities that would cause any alarm, however, and the effect of the pull on the rods was only barely appreciable at that section of the pier.

Alarm had been felt, and the opinion was freely ventured, that the pier was held standing by the weight of the trusses. To dispel this alarm, Case I was chosen; as might have been expected, it shows the fallacy of that opinion and the entire safety of the pier when standing alone.

The records indicated that the crib timbers were 2 ft. 6 in. apart, the old plans showing six longitudinal and thirteen transverse timbers in alternate courses, the bearing areas being the intersections shown cross-hatched in Plate XXXIII, a total of 78. The timbers were assumed to be 12 in. wide, thus giving a bearing area of 78 sq. ft. The diagrams showing the distribution of pressure on the timbers of the crib were prepared on this basis, on the assumption that the timbers carried the entire load, and the intensities that have been crossed out were the result. Several years later, when the crib was uncovered, it was learned that there were only five longitudinal and eleven transverse timbers in the alternate courses, and that the center intersection had been omitted in order to make room for the shaft, giving 54 intersections instead of 78. The timbers were found to be of various sizes, from 11 to 12 in. wide, so that the total bearing area was in reality about 50 sq. ft. It was also learned that the timbers carried practically all the load. Calculation with reference to the true conditions would have increased their intensity about 60%, or to the figures given above those crossed out. The maximum pressure of 555 lb. per sq. in. on one side, with 17 lb. per sq. in. on the other, calculated from the information available when the study was first made, easily accounted to the writer for the uneven settlement of the pier, but he had few adherents. Had the correct figure of about 888 lb. per sq. in. in cross-bearing been available at that time, all might have been convinced.

The position of the cross-section of the pier above the crib in Plate XXXIII was plotted from very careful measurements in 1908 from the plumb-line shown at the left. The position of the crib and caisson, shown in full lines, was estimated on the assumption that the tilt of the pier was caused by the crushing of the timbers uniformly distributed throughout the height.

The position which the crib and caisson would have occupied, on Mr. Patton's theory of settlement into the foundation and no uneven crushing of timbers, was found by projecting the sides of the crib at right angles to the top, the location and slope of which had been measured. The right-hand cutting edge of the caisson would have been at *C*, and if, by jacking under the other edge, the pier had been revolved about *C* to plumb position, it would have reached the position shown by the dotted lines. The absurdity of that theory is indi-

cated by the position the top of the pier would have occupied with reference to the pedestals of the trusses. The span to the right was never any farther to the right, its position being controlled by its proximity to the draw-span and the proximity of the latter to the south abutment. There would not have been room for the south pedestal of the next span to the north, shown at the left on Plate XXXIII.

The figures shown on Plate XXXIII convinced all concerned that there would be no danger in permitting the yokes to remain in place, and orders were issued to keep them tight. No fear was felt as to the effect of the pull on Pier 3, at the other end of the span, to which the yokes had been applied, because the south side of that pier had been built plumb, the entire batter being on the north side, the natural lean of the center of the pier being opposed to the direction of pull; in fact, the pull really improved the distribution of stress in Pier 3 in a manner somewhat similar to that at Pier 4, but less pronounced.

Fig. 3 shows Pier 4 after the yokes and I-beams had been applied, and on Fig. 14 there are transverse and longitudinal sections through the pier showing the relations between the yokes, the I-beams, and the pier.

Steel Bent Constructed at Pier 4.

In 1908, some one got the idea that it would be of advantage to provide further rigid support for the overhanging ends of the I-beams on the high side of the pier by building a concrete footing on the wide edge of the crib up to low water and placing a steel bent on top of that footing, as shown by Fig. 15. The I-beams under the pedestals were to be shifted far enough south to get bearing on the bent, and the spans were to be shifted as far south as possible in order to bring the load over toward the high side of the crib. This method might have been very effective had it been possible to move the spans any great distance, but, on account of the proximity of the south span to the draw, and of the north end of the north span to the edge of Pier 5, only a 6 to 10-in. movement could be made, and the effect on the line of pressure was barely perceptible. However, the work was done in the fall of 1908, by the Missouri Valley Bridge and Iron Company, at a cost of about $2 000. Fig. 16 gives a general idea of the manner in which the work was carried out.

Holes were drilled in the top of the crib and in the side of the pier at low water, and bent reinforcing rods were placed in them. A form

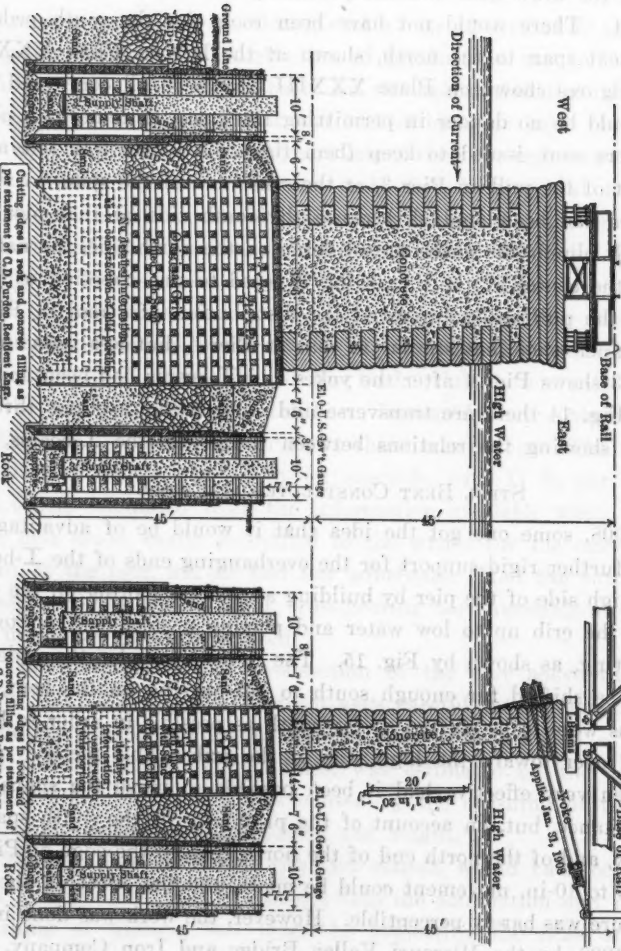
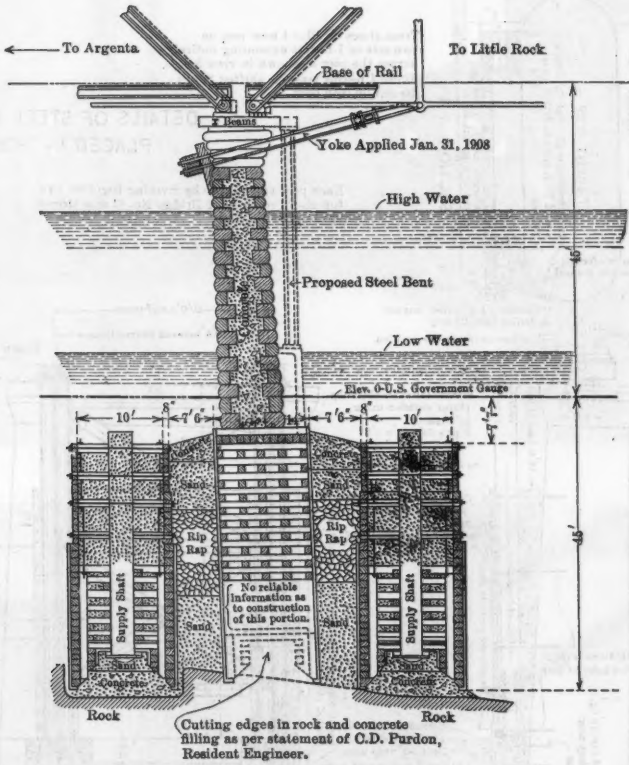

PIER 4
LITTLE ROCK JUNCTION BRIDGE
SHOWING CAISSON SUNK AROUND PIER IN 1898 AND 1899
AND SECTIONS AFTER PLACING I-BEAMS AND YOKES
West
East
Direction of Current
Base of Rail
High Water
Concrete
Ground Line
Rock
Sand
Gravel
Supply Shaft
Cutting edge in rock and concrete filling as per statement of C.D. Purdon, Resident Engr.
LONGITUDINAL SECTION
Fl. U.S. Gov't. Gauge
45'
To Argenta
To Little Rock
Irons
Yoke
Applied Jan. 31, 1900
20'
Irons 1" in 30'
SECTIONAL END PLAN

FIG. 14.

was lowered through the water to the top of the crib, and the concrete was deposited by a bottom-dump bucket. A steel bent, made of old top-chord sections, was then placed on this footing and held to the pier by U-shaped reinforcing rods set in grouted holes. The col-



SECTIONAL END PLAN
PIER 4
LITTLE ROCK JUNCTION BRIDGE
SHOWING CAISSON SUNK AROUND PIER IN 1898 AND 1899.

FIG. 15.

umns were then encased in concrete up to high water, and a reinforced concrete shell was constructed between them to ward off drift.

Then the I-beams and spans were shifted as far as possible, in order to throw the greatest possible load on the new bent. The work was carried out carefully and expeditiously, and offered no unusual

difficulties. Fig. 17 is an end view of Pier 4, showing the new bent in place.

Following this work, no appreciable movement in the direction of the bridge was apparent for several months, but the movement at

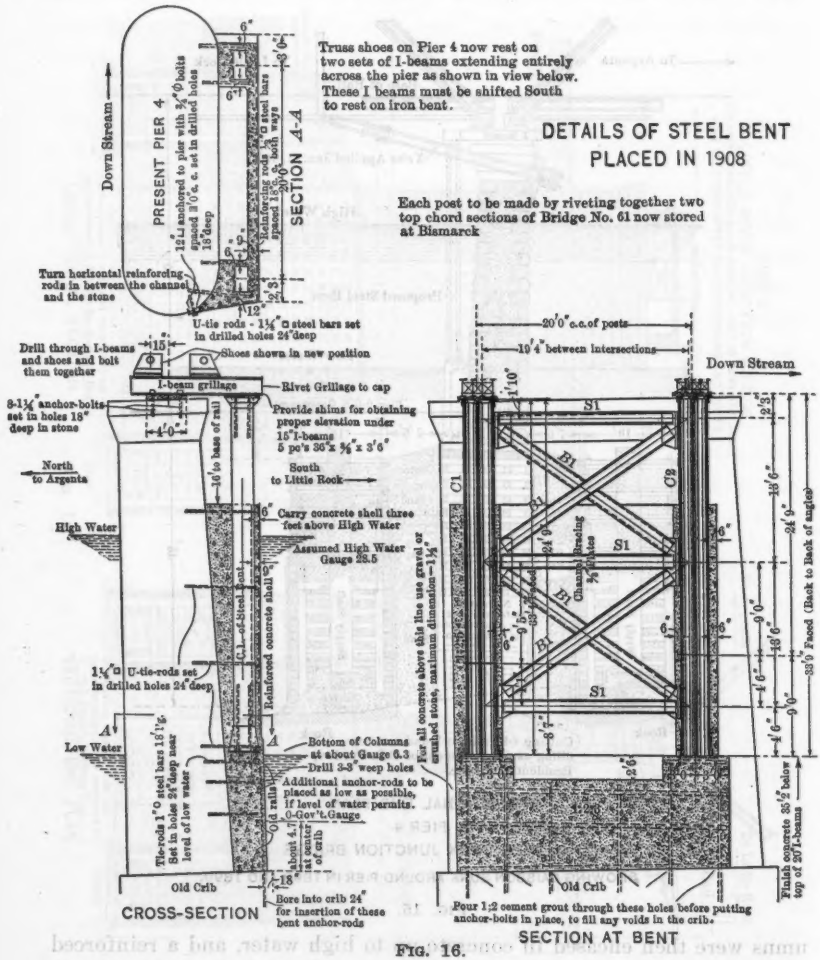


FIG. 16.

right angles to the bridge appeared to increase its rate, indicating that the work had no appreciable effect.

The movement of the pier was greatly dependent on the scour line. When the current shifted away from the pier, the bed of the

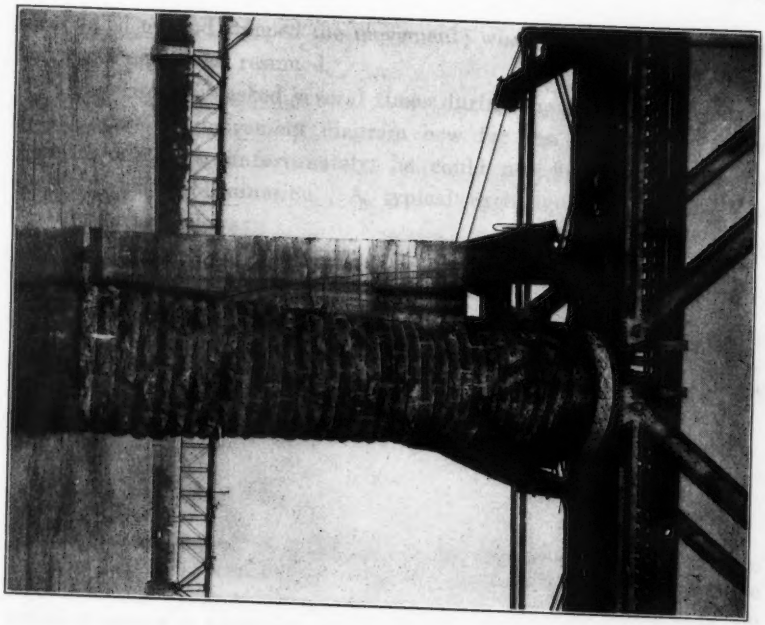


FIG. 17.—PIER 4, SHOWING STEEL BENT ENCASED IN CONCRETE.

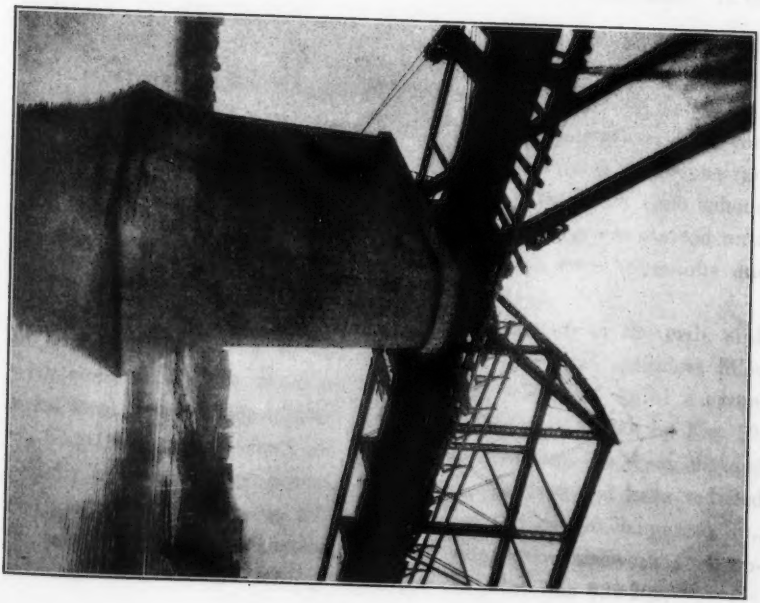
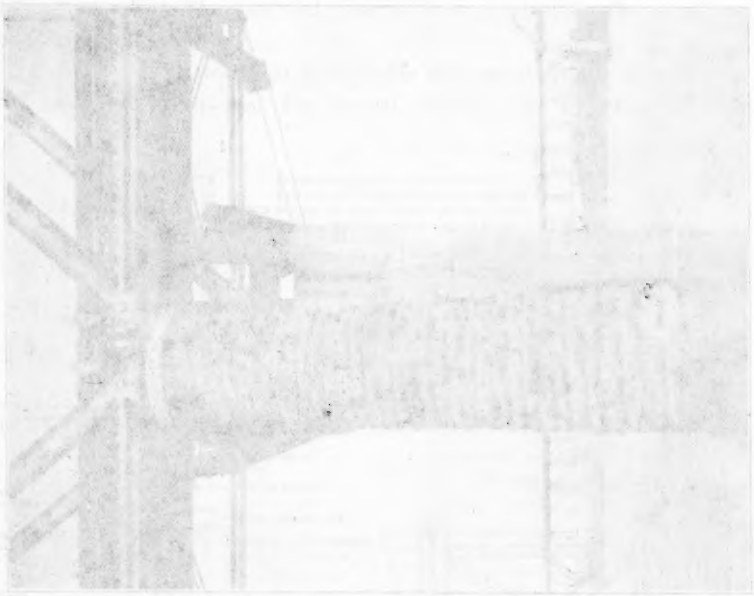


FIG. 18.—PIER 3 AFTER WORK WAS COMPLETED.



river filled up and stopped the movement; when the opposite was true, the movement was resumed.

The writer was asked several times during the next year or two to indicate on the movement diagram how far the pier could go and remain safe, but, unfortunately, he could not find any method of making that determination. A typical exchange of notes on that feature is as follows:

"ST. LOUIS, August 7, 1909.

"MR. HALE:

"I presume you are in touch with the movement records which Mr. Ford is keeping, and would like to have your opinion as to how far it is permissible to permit these movements to continue before taking further action.

"M. L. BYERS."

"8-17-09.

"MR. HALE:

"Movement record is attached. It is impossible for any one to say how far it is permissible to permit these movements to continue before taking further action. If the pier continues stationary no action whatever is necessary, as the piers are all perfectly safe. If the movement resumes we must take action.

"C. E. SMITH."

OTHER PLANS FOR OVERCOMING TROUBLE AT PIER 4.

Following the work in the fall of 1908, the observations were continued, and the matter of proposed permanent repairs or reconstruction was kept constantly alive for 3 years, during which time scheme after scheme was evolved and considered, but nothing was started until the fall of 1911. As a number of the schemes were ingenious and interesting, some of them will be described.

It was proposed to take down Pier 4 to the top of the crib, sink to rock along each side of the crib 12 or 15-in. steel cylinders filled with concrete, by the usual well-drilling methods, and build a structural steel tower on top of the cylinders and crib, as shown by Fig. 19. This was first studied on the basis of having the load divided equally between the cylinders and the crib, which would have reduced the maximum pressure on the timbers from 555 to 150 lb. per sq. in., on which basis the estimated cost, including the falsework for supporting the spans and taking down the old pier, was $32 200.

It was realized, however, that on account of the great stiffness of the cylinders and the steel tower, and the great elasticity of the crib, it would be impossible to control the distribution of load among the supports, and, in any event, most of the load would go to the cylinders. If the cylinders were made large and numerous enough and the steel tower strong enough to carry the entire load, the cost was estimated at $43 200.

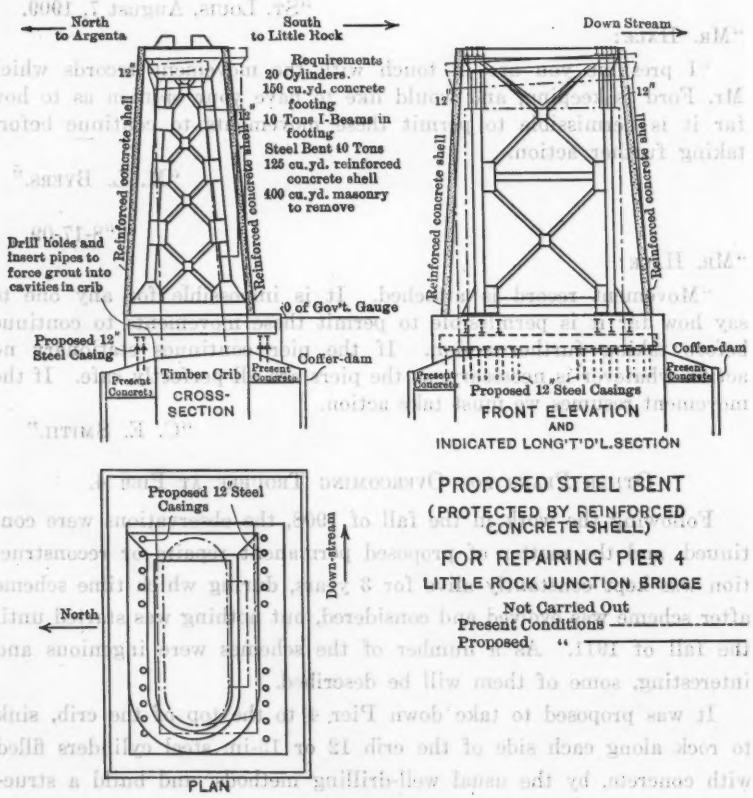


FIG. 19.

On account of the high cost and the uncertainties attendant on this scheme, it was abandoned.

The success obtained in placing the steel bent on the south side of the pier, and the small expense incurred, encouraged the proposition of building another steel bent on the north side. As there was no

projection of the timber crib on which to rest such a bent, and as the north edge was badly overloaded and thought to be crushing, it was proposed to sink a row of steel cylinders on the north side on which to rest the bent. This scheme also necessitated the use of longer and heavier beams on top of the pier, in order to distribute the load out to the bents. On account of the pedestals of the trusses being so far on the pier, it was almost impossible to study out any plan that would reduce the load on the pier an assumed amount, but, by changing the point of contact between the I-beams and the top of the pier, it was found possible to throw about 250 000 lb. on the new bent, thereby reducing the maximum pressure on the timbers from 555 to 477 lb. per sq. in. This was estimated to cost about $10 000. The scheme was abandoned on account of its uncertainty.

Of course, it would have been possible to have taken the weight entirely off the old pier and provide cross-girders sufficiently strong to throw the entire load on the steel bent, but the extra expense and uncertainties of this scheme ruled it out.

In the meantime the writer had worked up the plan shown on Fig. 20, which contemplated driving piles to rock by jetting, within two sides of the annular space between the crib and the coffer-dam, toward which the pier was moving, the plan being to remove the concrete cap a section at a time, drive several piles, and replace the concrete as a sub-footing course, and incidentally to preserve the side support afforded by the concrete. To accomplish this it was proposed to build a puddle coffer-dam around the pier within the 1898 coffer-dam in order to hold the water out against about a 10-ft. head, and to force as much concrete and grout as possible into the crib.

After grouting the crib, driving the piles, and restoring the concrete cap, it was proposed to construct, around the old pier and on top of the projections of the crib and on the new piles, a solid reinforced concrete structure which would materially enlarge the base, restore the symmetry, and effectually protect the masonry for all future time.

The rough rock-face and round ends of the pier made it particularly suited to encasement, but, in order to make the bond as intimate as possible, it was planned to drill through the body of the pier at several places near the bottom and place second-hand eye-bars through the pier, with suitable anchors at each end to be embedded in the con-

crete shell; also to curve the horizontal reinforcing rods continuously around the ends of the pier, and to replace the top by a new thick reinforced concrete cap. This plan had the advantage that no false-work would have been necessary, and it could have been carried out for $12 000.

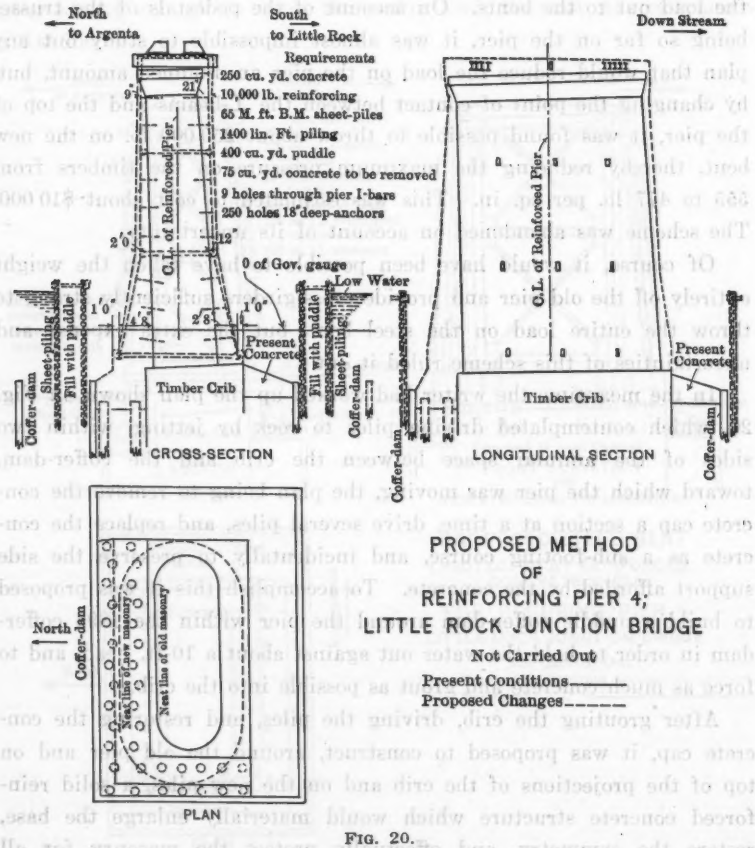


FIG. 20.

The writer has always felt that this method of repairing Pier 4 was the best that could have been devised, but so many objections were raised, as to the difficulty of extending the old coffer-dam up to make it water-tight, the danger to the pier of removing the concrete cap placed in 1899, the difficulty of driving the piles, and the possible damage to the pier by the adjacent pile-driving, the danger of

boring holes in the old crib for grouting (and possibly letting the sand run out), the impossibility of getting proper bond between the new and old work under traffic, the danger of disturbing any of the former conditions without carrying the spans on falsework, etc., *ad infinitum*, that the scheme was dropped.

Proposed Falsework for Supporting Spans.

Strange to say, several engineers who apprehended that if any work were done on the pier it would fall down, and desired the spans placed on falsework to save the steel spans in case of the collapse of the pier, proposed to support the spans in such a way that if the pier did fall it would knock the supports out from under at least one span and cause that to fall. When the writer pointed out this condition, he was requested to arrange for such temporary support of the spans resting on Pier 4 as would hold them up in case the pier should fall, the idea being to have plans for such temporary supports thoroughly understood by and in the hands of all concerned, with authority for the division forces of the Railway Company to erect the supports immediately on the further rapid movement of Pier 4.

FALSEWORK FOR SUPPORTING TWO SPANS OF LITTLE ROCK JUNCTION BRIDGE CONTEMPLATING THE REMOVAL OF PIER 4

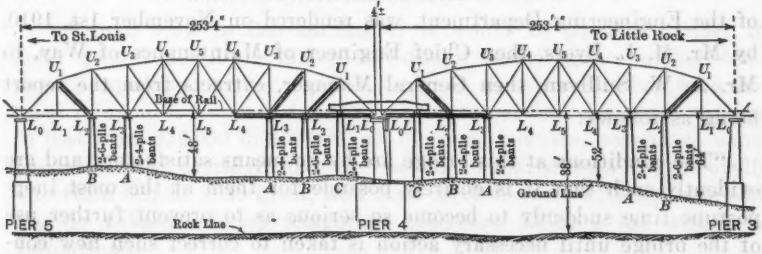


Fig. 21.

Fig. 21 shows the general plan that was worked up. On account of the necessity for retaining the greatest possible clear waterway, and the impossibility of maintaining any longitudinal bracing below high water except with disastrous results, it was proposed to place five double bents under each span, and to bridge over the pier with through girders, from which the floors would be hung. In the event of the unexpected happening at Pier 4, it was proposed to erect bents, *A*, under the third panel point on each side of Pier 4 and stiffen the tension members in the ends of the trusses so as to enable them to

resist the compression that would result if the pier were removed. It was expected that when the spans had been supported in this way, traffic could be suspended while the pier was being taken down to prevent its collapse, if the emergency should require; or, the spans having been protected against the collapse of the pier, traffic could be maintained until its collapse, and the remainder of the falsework could be put in in the meantime. The material for two double bents and for stiffening the overhanging ends of the trusses was assembled at the bridge and held there for several years, until after the reconstruction of Pier 4.

Further Studies.

Continuing into 1910, the entire matter was studied from every conceivable standpoint, including reconstruction of all the piers as single-track in their old location, reconstruction for double-track, an entirely new set of piers and shifting the spans, entirely new single- and double-track bridges, and rearrangement of the method of operation of the Little Rock Terminals so that the bridge could be abandoned, all traffic to use the main-line bridge. None of these plans was carried out, but the "watchful waiting" was continued.

A full report on this situation, with an expression of the attitude of the Engineering Department, was rendered on November 1st, 1910, by Mr. M. L. Byers, then Chief Engineer of Maintenance of Way, to Mr. A. W. Sullivan, then General Manager, extracts from the report being as follows:

"The conditions at this bridge are by no means satisfactory and are evidently such that it is entirely possible for them at the most inopportune time suddenly to become so serious as to prevent further use of the bridge until necessary action is taken to correct such new conditions. This may occur at a period of high water, which would prevent the use of the bridge for some six months to a year, although I do not believe there is serious danger of the loss of any of the superstructure of the bridge. It would probably be possible at any time (unless the failure were extremely sudden, which is unlikely) to drive such falsework as would be necessary to support the two spans resting on this pier, although in this case train service would have to be discontinued.

"There are a number of factors entering into the decision as to what should be done at this point. It may be that considerable time will be required to carry out the plans which may ultimately be de-

cided upon. It is impossible to determine how long the present conditions may continue. It may be that they will change almost at once, or it may be that they will not change for years. It would seem desirable, therefore, if possible, to decide upon a plan of action in case of sudden change for the worse so that no time will be lost in the preparation of plans or in necessary further study of the situation. To this end the following is submitted for your consideration:

"An entirely new set of piers for a single-track bridge, either in the present location or at one side of the present location, would cost approximately..... $250 000.

"An entirely new set of piers for double-track bridge, either in the present location or to one side of the present location, using the present spans, would cost approximately.. $450 000.

"A new double-track bridge (including the piers) would cost approximately..... $800 000.

"The present Baring Cross Bridge (at *Y*, see Fig. 2) will probably require renewal within five years at an estimated cost of approximately (for double-track)..... $900 000.

"In the event of the failure of Pier 4 alone, the probable cost of entirely rebuilding it for single-track would be about ..... $60 000.

"In the event of the failure of Pier 4 alone, the probable cost of entirely rebuilding it for double-track would be about ..... $80 000.

"With a view to determining if it would be possible further to defer final renewal of this pier through resorting to further temporary expedient, a very careful study of the situation has been entered into and a number of tentative schemes worked out, these varying in cost from $12 000 to $50 000; all of them, however, present so many possibilities of casualty, and the cost as compared with the decidedly uncertain benefit is so great, that none of them meet with my approval, nor do I know of any temporary expedient which I am willing to recommend.

"As before stated the further period during which the present temporary expedient can be continued is entirely uncertain and there is the possibility that it may fail on extremely short notice.

"Figure 8 is a general elevation of Pier 4 and the two spans resting thereon and shows in red the temporary falsework necessary to be put in place in order to prevent these spans falling into the river in case Pier 4 falls.

"Figure 9 shows the character of the temporary piers and the two plans indicate the amount and character of the work which must be done in order to save the spans in case of the falling of Pier 4. It must be borne in mind that it is quite possible this work, if it becomes

necessary, must be carried out at the time of extreme high water, and it is absolutely impossible to foretell the amount of time that will elapse between the time when the rapid movement of Pier 4 is detected and the time when the pier will actually fall—we may have time to place this falsework and we may not. The falsework has been ordered and will be kept on hand at Argenta.

"Figure 8 shows in yellow the additional supports that must be provided in case of the loss of Pier 4 before the bridge can be again opened to traffic. During the period of high water there will, of course, be more or less amount of danger that drift or boats would destroy this falsework—such is the risk involved in the present conditions.

"Quite recently Pier 3 has begun to move longitudinally at a much faster rate than heretofore, and it is, of course, possible that the conditions with which we are confronted at Pier 4 may at any time have to be met in connection with movement of the other piers.

"Recommendations.

"In view of the varying circumstances above enumerated, it is my recommendation that it be decided at once what policy is ultimately to be followed in connection with this bridge. I believe all of the facts essential to this determination are stated in this report and the matter would seem to be one of policy rather than engineering.

"Yours truly,

"M. L. BYERS,

"Chief Engineer, M. of Way."

PROPOSED MAT AROUND PIER 4.

For two or three years prior to 1911, the channel of the river had been moving away from Pier 4 and concentrating under the draw-span, resulting in the building up of the river bed around this pier with clear fine sand which early in 1911 covered the top of the crib and reached up on the masonry within 1 or 2 ft. of extreme low water. This sand evidently filtered into the crib and gave increased supporting power in addition to that provided by the sand that enclosed it. As a consequence, the movement of Pier 4 gradually slowed down, so that it was barely perceptible, and the great uneasiness that had prevailed for several years was partly allayed. The decrease in the movement was attributed to the steel bent placed at the back of the pier, by those responsible for that monstrosity.

In order to protect and preserve this sand surrounding the pier, the writer attempted in the fall of 1910 to secure authority to spend about $2 000 to construct a standard brush and pole mattress, about

150 ft. square, around Pier 4, to be securely bound together with wire and wire strand, and covered with rip-rap placed in pockets of the mattress to be formed by placing two sets of poles one across the other on top of the mattress to prevent the rip-rap from rolling off.

At the low stage of the river following the request for authority, the work could have been done without barges or other floating equipment and brought to completion in 30 days. The authority was not granted, and, after the next flood, it was not needed, as much of the sand had been washed away.

Rapid Movement of Pier 3.

While the river bed was building up around Pier 4, it was scouring away around Pier 3, the north rest pier of the draw-span, and this pier started to move quite rapidly directly up stream, at right angles to the axis of the bridge.

The movement in 1911 was more rapid and constant than any experienced previously in the other piers, and indicated the necessity for immediate action. The pier did not move dangerously in the direction of the bridge. Its natural tendency would have been to move south, as the plumb south face and eccentric bearings of the spans made the pressure heaviest on the south side. It was restrained from movement in that direction, however, by the rods and yokes which made this pier an anchor for the pull of Pier 4. In fact, during the frequent tightening of the yoke rods, Pier 3 was observed to move slightly north into a more erect position.

Plans for Reconstruction of Pier 3: Modjeski's Recommendation.

The first plan prepared for the reinforcement of Pier 3 was somewhat similar to the writer's last plan for Pier 4, which had been rejected. (See Fig. 20.)

It consisted of a reinforced concrete shell, tied to the old pier and resting on additional piles to be driven to rock around the old crib, as shown by Fig. 22. Here, however, there was no surrounding cofferdam, and it was deemed advisable to furnish protection for the piles and additional stability for the pier.

This was to be obtained by sinking around the pier mud-cell mattresses in layers about 4 ft. deep, with enough rip-rap in them to sink them. The result would be a building up of mattresses of

decreasing sizes and rock; sand sediment would immediately deposit until the top of the crib was nearly reached, when the entire mass would be covered by a heavy layer of rip-rap, the latter and the surrounding bed of the river would then be covered by another large protecting mattress. It was also the intention to bore holes in the crib and get in as much grout as possible. The writer believes this method of repair would have been entirely satisfactory; it also had the advantage of being very cheap. Five bids were received, the lowest (from an entirely reliable contractor) being about $8 500, which, together with the extras and incidentals, would probably have made the total cost of the work $10 000.

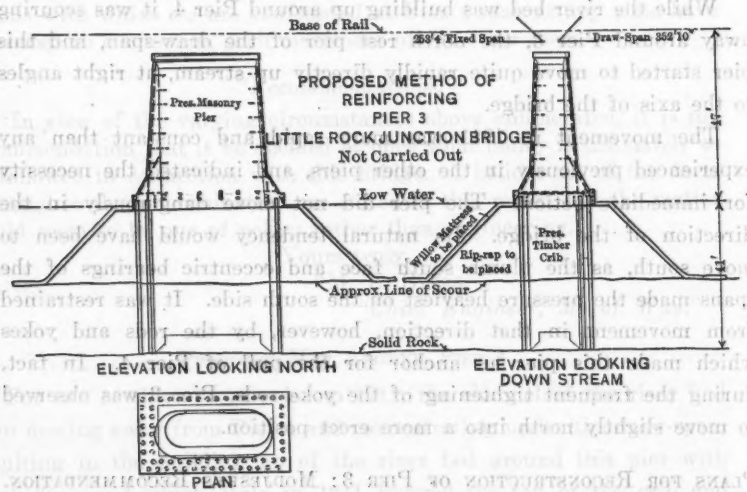


FIG. 22.

Before proceeding along these lines, however, the management desired an expression of opinion on the entire problem by a disinterested engineer, and Ralph Modjeski, M. Am. Soc. C. E., was consulted. The writer disclosed to Mr. Modjeski all the information in possession of the Railway Company on this subject, and accompanied him on an inspection of the bridge.

Copies of Mr. Modjeski's reports of August 5th, August 7th, and August 19th, 1911, are given in Appendix D.

It was arranged between Mr. Modjeski and the writer to bore holes down through the crib to rock, and this was done as far as

possible. It was found to be very difficult to get the pipe down; it would encounter solid timber, then sand, then loose rock, and combinations of all these. However, it was finally driven to the solid timber roof of the caisson, which was encountered at such an elevation as to leave no doubt that the cutting edge rested on rock, the level of the latter having been ascertained by running a jet pipe down through the sand around the pier. In sinking the pipe through the crib, efforts were made to secure the services of expert well drillers, but they all refused to guarantee to get the pipe down.

In brief, Mr. Modjeski recommended first that the sand in the crib of Pier 3 be removed and the crib filled with concrete. To accomplish that, he recommended driving 50-ft. triple-lap sheet-piling entirely around the pier, 6 ft. from the crib, and examining the old crib by partly removing the old planking. He also recommended that the up-stream end of Pier 3 be supported on shores resting on piles driven alongside the old cribs. This work involved supporting the spans on falsework.

A later report suggested placing the coffer-dam 12 in. from the old crib (if it was found that the filling of the cribs was sand which could be pumped out), excavating the material between the old crib and the coffer-dam, and sealing the bottom with concrete. Then the top crib timbers and the material inside were to be removed and the crib refilled with concrete, this work necessitating the support of the spans on falsework and taking down the old pier.

Later, he recommended rebuilding Piers 3 and 4 entirely by supporting the spans on falsework, putting coffer-dams around the piers, taking down the masonry, filling the cribs with concrete, and building new piers on them.

After considerable study of these latest recommendations, it was tentatively decided to follow Mr. Modjeski's first recommendation, but, on account of the great head against which pumping would have to be done (up to 50 ft.), steel sheet-piling was contemplated. Detailed estimates, however, indicated that this method would be quite expensive and uncertain, and would involve the hazard of carrying the spans on falsework. As a slight August rise in this river, a few days before, had swept away 1 000 lin. ft. of falsework at a bridge about to be erected farther up stream, those responsible for this bridge were not enthusiastic about carrying the spans on falsework.

Taking down the piers to rebuild them entirely would have been fully as hazardous, and very expensive.

Mr. Modjeski was then requested to pass opinion on the writer's plan of driving piling around Pier 3 and resting on it a reinforced concrete encasement around the pier, protecting the piling with mud-cell mattresses and rip-rap, as shown by Fig. 22. This he objected to as somewhat uncertain, as to results and the eventual stability of the pier.

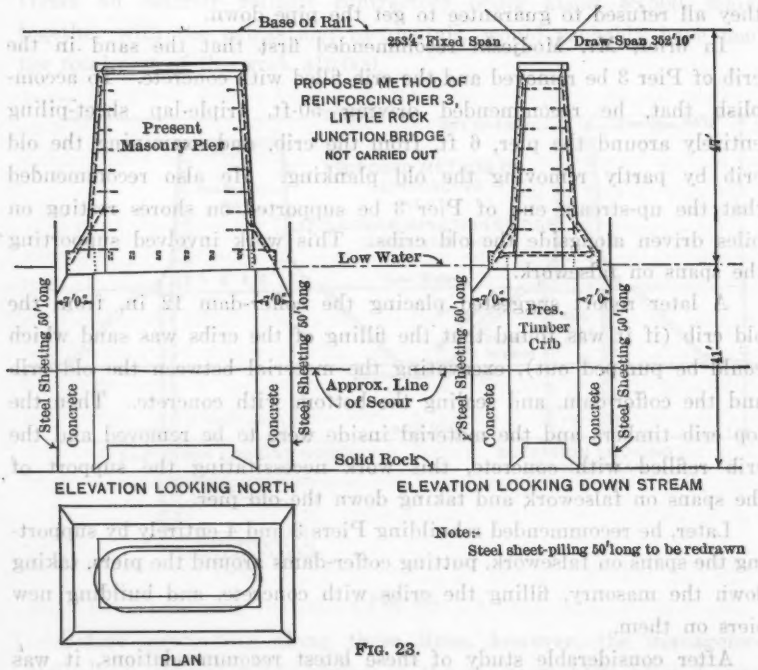


FIG. 23.

He was also asked to pass on a plan for driving steel sheet-piling about 6 ft. from the old crib, excavating the space between it and the old crib, and filling that space and the crib with concrete, the pier to be encased in a reinforced concrete shell, the whole to be done without falsework. Fig. 23 shows this scheme. He pronounced it to be very expensive, and said:

"To attempt to carry out this scheme without supporting the spans on falsework would seem to me extremely risky. The excavation around the crib would deprive it of considerable skin friction and would

probably open up more places through which sand now in the crib could leak out. The crib is now settling at the rate of 1 in. per month by the crushing of the timbers and this settlement may under the new conditions get so rapid as to endanger the whole structure.

"I am convinced that the pier could be built new for less money than the schemes shown on your plan. The new pier would probably contain about 500 cu. yd. of concrete and there may be another 500 or so necessary to fill the crib, making, say, 1 000 cu. yd. of concrete in all, besides, the work would be more certain of accomplishment and with the least amount of unforeseen expensive contingencies. I would therefore strongly recommend that this pier be rebuilt as stated in my former letter to Mr. Pearson. While it is very difficult to make even an approximate estimate on the cost of such work, my opinion would be that the new pier would cost very little more, if any, than the cheaper one of the two schemes in question, assuming in all cases that it would be desirable to support the adjacent spans, as there is great uncertainty how the present crib would behave while any work is being done around it.

"To rebuild the pier I would proceed as follows: Build temporary supports for the span, carrying the ends on a set of girders. Take down the old pier to the crib work. Drive wooden triple-lap sheet-piling or steel sheet-piling around the present crib. Excavate present crib and fill with concrete by methods which will become more apparent as the work goes along. Build the new concrete pier on top of the crib work thus reinforced."

The method of reinforcement that was finally brought to successful conclusion was first suggested by E. J. Pearson, M. Am. Soc. C. E., First Vice-President of the Railway Company. To overcome all uncertainties, he suggested securing the necessary additional supporting power by the use of a pneumatic caisson on each side of the old crib. This was extended, later, by adding caissons for the ends, forming an annular caisson surmounted by a coffer-dam, the space above the annular caisson to be filled with concrete, doing the same to the crib, and encasing the old pier in a new reinforced concrete shell, as shown by Fig. 24. This was also referred to Mr. Modjeski by the writer, in a letter setting forth the plan, as follows:

"I have never fully abandoned the idea which you first proposed of filling the pockets within the crib with concrete, and think if this work can be accomplished that the problem will be solved. I find upon investigation that it will cost very little more to sink an annular pneumatic caisson entirely around the crib than to carry out the scheme formerly proposed to use steel sheet-piling and concrete filling.

"I hand you herewith blue print, showing in red the construction of the caisson. I would construct the working chamber entirely of steel, as this can be done, by the use of second-hand bridge members now on hand at our store yard, for very little cost. I would leave off the inside wall of the coffer-dam above the roof of the working chamber, and construct a water-tight outer casing wall braced by latticed steel members extending up to low water.

"After sealing this working chamber with concrete, this outer shell would form a coffer-dam entirely around the pier.

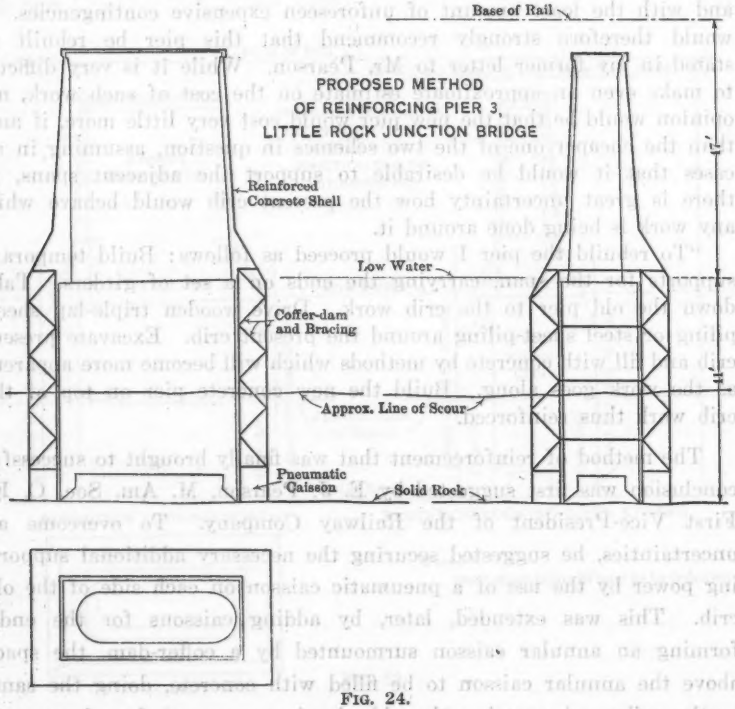
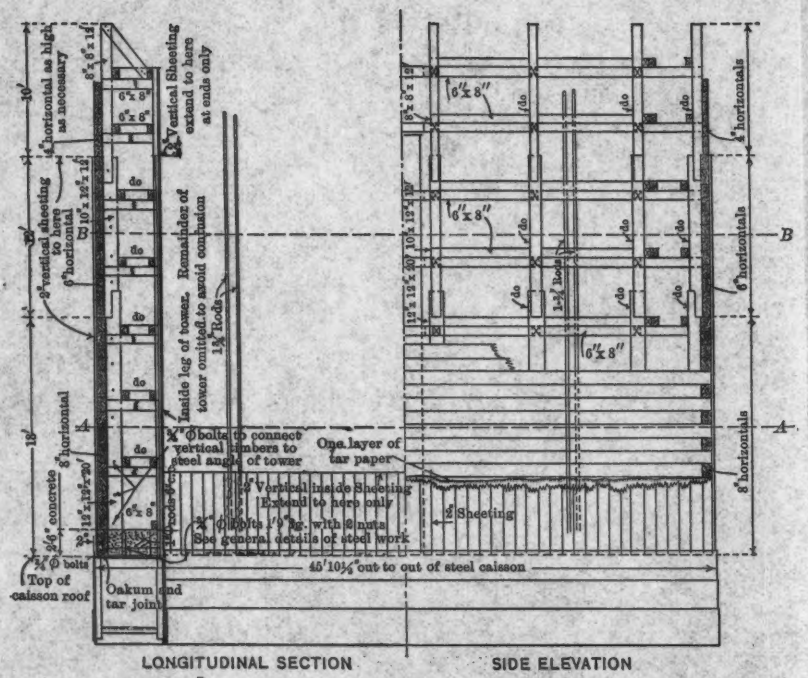
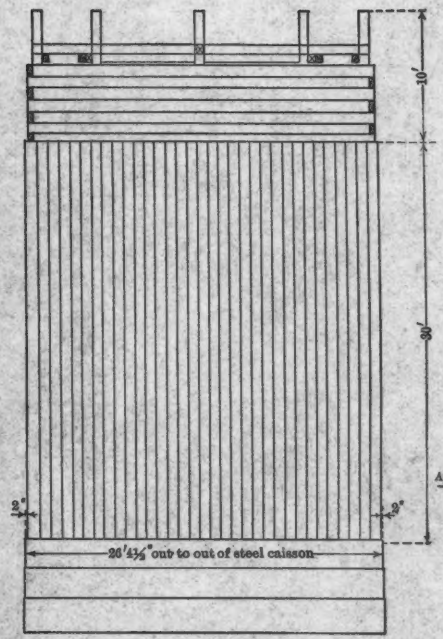


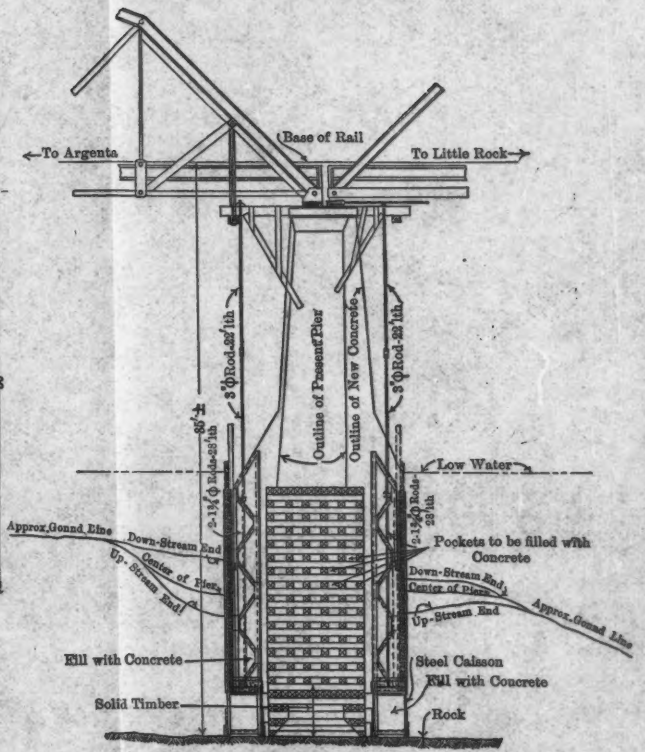
FIG. 24.

"The Arkansas River is subject to very rapid rise and excessive scour, and at such times carries an immense amount of drift. It is a hazardous proceeding at any time to carry spans on falsework in this river. During the construction of our Baring Cross Bridge, I understand, one of the spans was lost on this account. The Arkansas Bridge Company recently lost 1 000 lin. ft. of falsework in this same river a few miles above Little Rock.

"For this reason I do not like to place spans on falsework and take down the pier, except as a last resort.

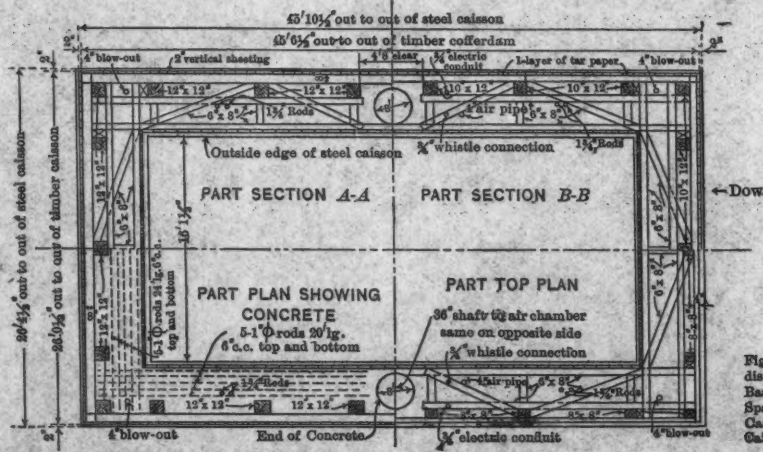


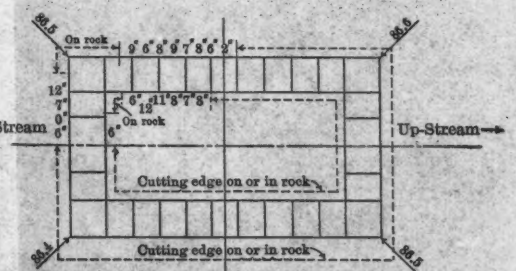




CROSS-SECTION OF PIER 3

NOTE.
When Grillage under old Pier was uncovered it was found to be made up of 5 Longitudinal and 10 Transverse courses, (below the 2 top solid Courses)





Figures along sides show distance of cutting edge above rock. Figures at corners show distance of cutting edge below Base of Rail at Pier 3 which is 3½" lower than Base of Rail at Pier 2 (Pivot Pier)
Spaces between cutting edge and rock packed with neat cement in bags
Caisson landed at final elevation Feb. 25, 1912
Caisson filled with concrete Feb. 29, 1912

SKETCH SHOWING FINAL ELEVATION OF STEEL CAISSON AROUND PIER 3

DETAILS OF TIMBER COFFER-DAM FOR USE IN REPAIRS TO PIER 3, L.R. JC. BRIDGE

UNIV. OF MICH.

"If the pier starts to show signs of more rapid movement during the sinking of the caisson, traffic could be removed from the bridge temporarily and the sinking proceeded with. The pier could then be supported by temporary struts and shores resting on the steel members above the working chamber. After the caisson were sunk to the full depth and the working chamber sealed, I would pump out around the present crib and ram the pockets full of concrete, using a cement gun for the purpose, if necessary, and working from both sides of the crib. I would then complete the work by completely filling the space above the working chamber with concrete, and would then encase the present masonry pier with reinforced concrete above low water, both for the purpose of securing a symmetrical appearance and also to distribute a considerable portion of the load to the caisson."

Mr. Modjeski thought that this plan would be unduly expensive and, if carried out without falsework, entirely too hazardous. As he did not disapprove the scheme, however, it was adopted and carried out successfully.

Reinforcement of Pier 3 in 1912.

Unfortunately, the delay in arriving at a decision as to what would be done at Pier 3 consumed the best months of the year for the work. The change to pneumatic construction, necessitating the assembling and erection of the pressure plant and the construction of the caisson, further delayed the work up to the time of the flood period. The compressor plant was finally ready, and the steel caisson was shipped early in November.

Plate XXXIV shows the general outlines of the annular caisson and coffer-dam. The old records showed the old cassion under Pier 3 to be 14 by 31 ft., so the dimensions of the inner cutting edges of the new caisson were made 16 by 33 ft. The caisson was made of steel primarily to reduce its necessary width and consequently the displacement; the presence in the old bridge-material yard of a large number of duplicate second-hand floor-beams which could readily be converted into a caisson also encouraged the use of steel. The details of the steel caisson and the joint between its roof and the coffer-dam are shown on Plate XXXV. It was decided to build up a coffer-dam on the outer wall of the caisson, connecting the coffer-dam to steel lattice work or towers bolted to the roof of the caisson. These towers were provided in order to form the backbone of the rigid framework it was known

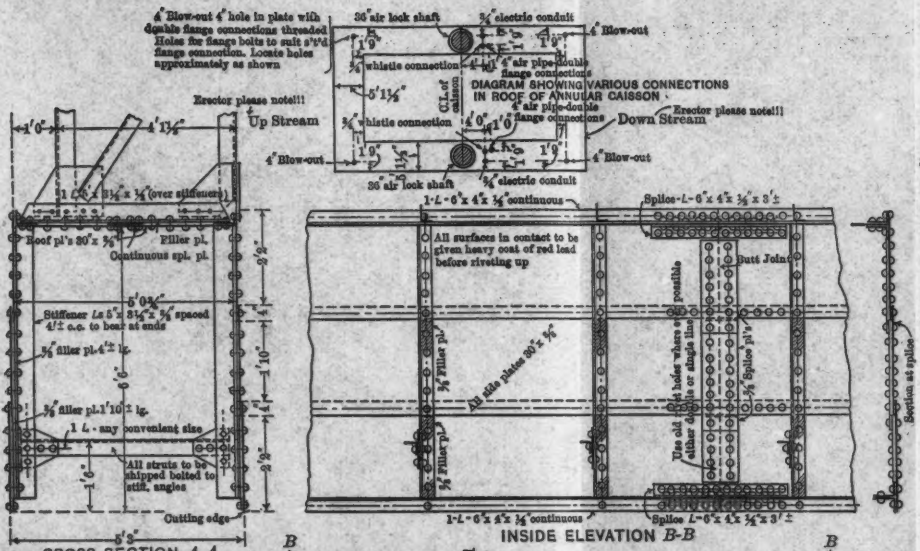
would be required to brace the new coffer-dam against the old crib during excavation.

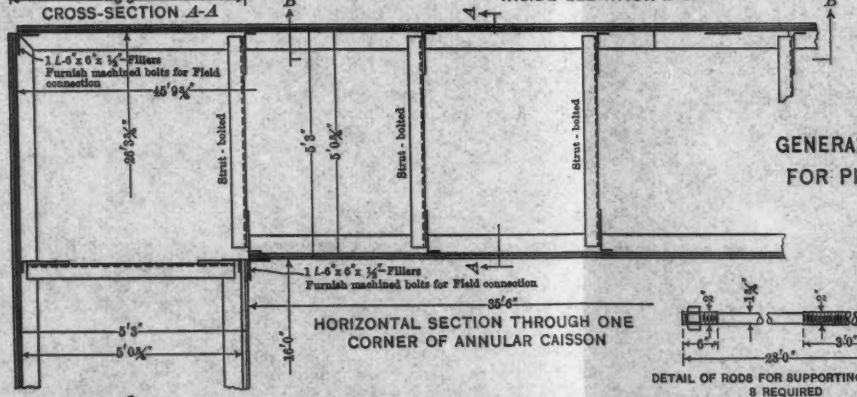
The greatest care was taken in the design of the joint between the caisson and the coffer-dam. The timbers in the bottom row, 8 in. thick, were cut to fit the steel as closely as possible, and were set in a bed of hot tar and oakum. Surfaced timbers were used, and each layer was set on the next below in hot tar and caulked. Tar-paper was then placed over the timbers and 2-in. matched sheeting placed outside. The lower ends of the sheeting were driven into a groove, filled with hot tar and oakum, over the roof of the caisson, formed between the bottom timber and the upstanding leg of the angle at the top corner of the caisson. A space about ½ in. wide, between the sheeting and the angle, was then caulked. To bind all parts together effectually and to provide additional weight for sinking, a 2 ft. 6-in. layer of concrete was then placed over the roof of the caisson enclosing the bottoms of the towers. This concrete was banded to the steel roof by a large number of ¾-in. bolts, 21 in. long, fastened to the steel plate and extending vertically into the concrete.

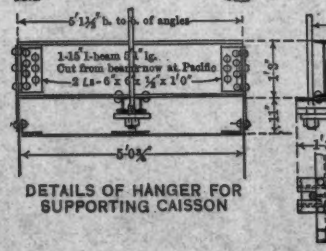
The construction of the remainder of the coffer-dam was similar to that just described.

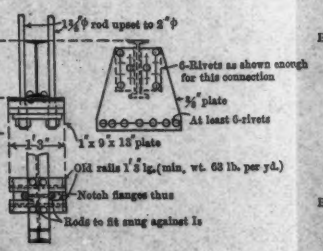
For the support of the annular caisson during the assembling and sinking, until the entire load rested on the river bed, the temporary support from the pier and trusses shown on Plate XXXVI was provided. Two 8 by 16 in. by 28-ft. timbers were placed across each end of the pier, as close as possible to the truss shoes. The coping stones at each end had to be removed in order to place the timbers low enough to permit the draw-span to swing over them. The ends of the timbers were held up by inclined struts gained into their under sides and set in grouted notches cut into the masonry of the pier. For additional stability and stiffness, the north end was connected by rods and struts to the end posts of the north span which, in turn, were stiffened by timber struts (similar to collision struts) running down to the first panel points. On account of the necessity of having the draw-span free to swing, it was not possible to provide corresponding connection with the end post on the south side of the pier.

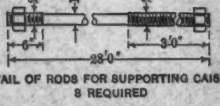
Four 3-in. steel rods, threaded for their entire length, were passed through between the ends of the timbers, the nuts on them resting on heavy plates over the timbers, and the lower ends of the rods

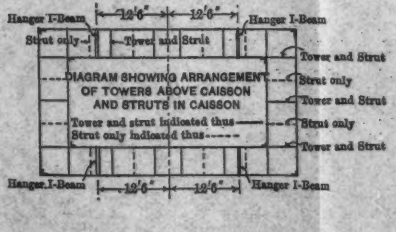


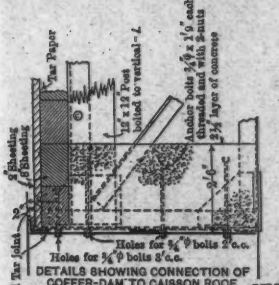


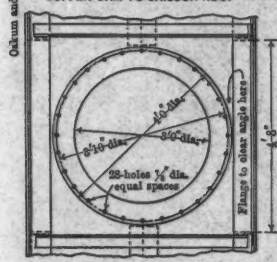


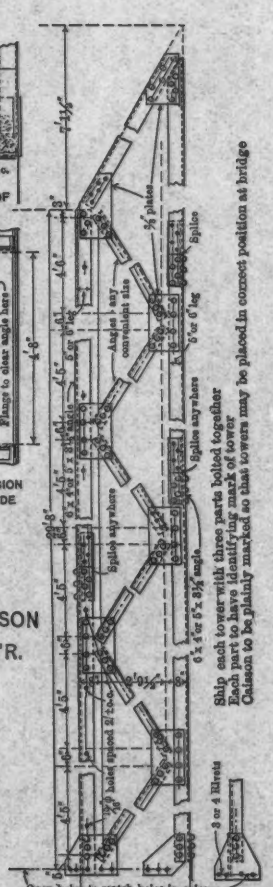












GENERAL DETAILS OF ANNULAR CAISSON FOR PIER 3, LITTLE ROCK JUNC. B'R. LITTLE ROCK—ARK.

Ship each tower with three parts bolted together
Each part to have identifying mark of tower
Caisson to be plainly marked so that towers may be placed in correct position at bridge

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reaching down to chairs made of rails supporting other smaller rods in pairs, on which the long sides of the caissons were supported near the water level after being floated out to the pier. The short sides or ends were then floated on barges and connected to the long sides by machine bolts. After the caisson was assembled and caulked, and all shaft, pipe, and wire connections were made, it was lowered into the water and the construction of the coffer-dam was started.

During and after the placing of the 30-in. layer of reinforced concrete over the roof of the caisson, the sinking progressed as fast as the work permitted. At the commencement of sinking, the river bed at the down-stream end of the caisson was several feet higher than at the up-stream end. As the caisson was lowered, the sand scoured deeper and deeper at the up-stream end and the workmen shoveled the sand from the down-stream end of the caisson into the water at the up-stream end until the cutting edge landed all around, following which the sand and other material, much of which consisted of heavy rip-rap and logs (almost petrified), was removed in buckets through the locks. The removal of the sand formerly against the down-stream end of the pier relieved it of pressure to such an extent that it partly righted itself. Trouble was continually experienced by the caisson catching on the protruding upper ends of the sheeting planks of the old crib, which had been torn loose by the settlement of the timbers and bulged by the pressure of the material escaping from the crib. These were particularly annoying before the cutting edge rested on the river bed, as the workmen had to pick their footing in the caisson. On all the old records available the caisson of the old pier was shown to be plumb, and a clearance of 2 ft. was supposed to have been allowed between it and the new caisson, but it was found that the old caisson had a sharp batter, which necessitated ripping off the old vertical sheeting before the new caisson could be landed. The cutting edge of the old pier was found on rock everywhere except at short dips which had been filled with concrete.

All the material within the new caisson had to be removed through the locks because sufficient pressure could not be maintained to force the sand through the blow pipes. This was caused by all the material between the old and new work running into the caisson and leaving the inner cutting edges usually exposed; and, when an attempt was made to increase the pressure to blow out the material,

the air would blow out under the inner cutting edge instead. As it was desired to avoid the resulting washing out of sand from the old crib caused by the commotion in the water during these blow-outs, the efforts to use the discharge pipes were abandoned. Mattson locks were used, and permitted the rapid removal of the material. No other particular difficulties were encountered.

The cutting edges were everywhere landed on rock, except over short dips, which were cleaned out and filled with concrete. The working chamber was filled with concrete carefully rammed to the roof, and before it had set, the lower sections of the shafting were also filled with grout.

Fig. 25 shows the steel caisson after it had been lowered partly into the water and construction of the coffer-dam had commenced. Fig. 26 shows the upper edge of the coffer-dam after the caisson had been landed and the reinforcing rods for the concrete encasement were being placed.

While the caisson was supported from the pier the inspector was required to compute daily and report weekly the weight supported. To guide him in the control of the work, and to avoid overloading the temporary supports, he was told the maximum weight that could be allowed, and was given a copy of a typical programme of work and a tabular summary of weights, together with the following instructions, the whole idea being to control the construction and sinking in such a manner as to keep the smallest load on the rods and take advantage of the immersion:

"Instructions.

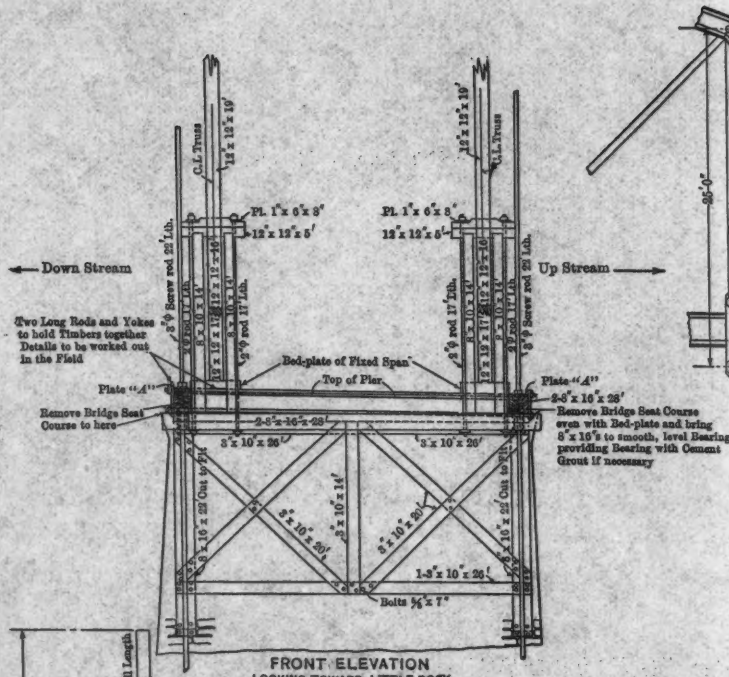
"The support that has been provided for lowering the caisson into its final position is capable of safely supporting a total load of 325 000 lb. and the total weight of the caisson and coffer-dam and other materials above the caisson suspended from the rods should not exceed the above weight.

"There are five conditions that must be met in the sinking and founding of this caisson.

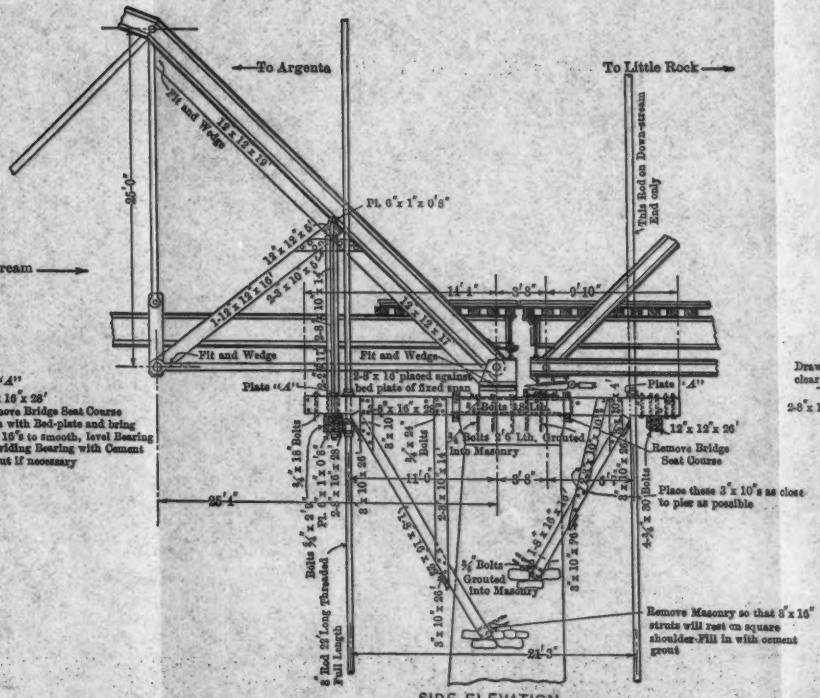
"They are as follows:

"1. Total weight of structure before lowering into water must not exceed the strength of the supporting rods.

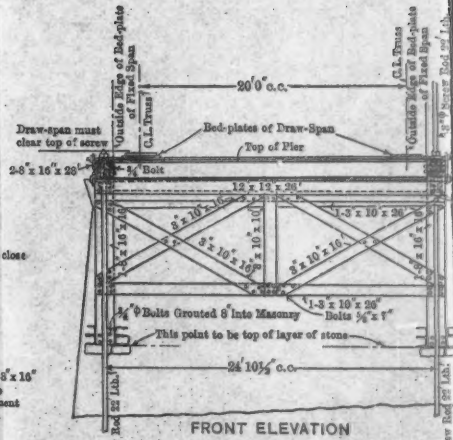
"2. Total weight of structure decreased by the buoyant effect of the water displaced during sinking must not exceed the total supporting power of the rods.



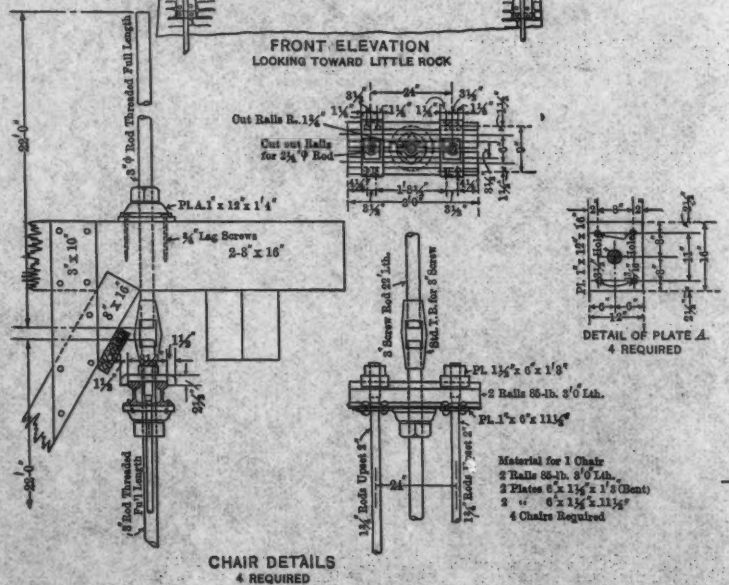
FRONT ELEVATION
LOOKING TOWARD LITTLE ROCK



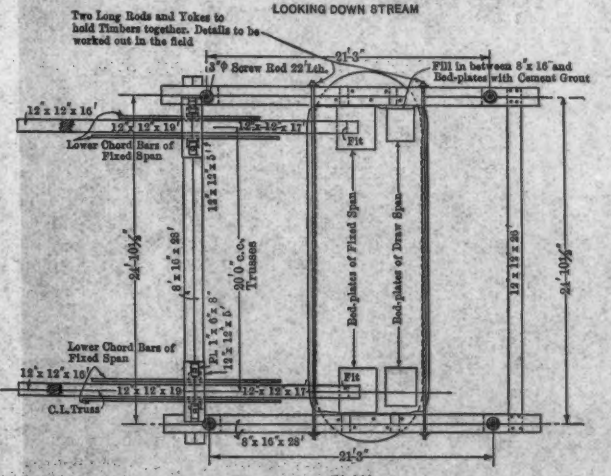
SIDE ELEVATION
LOOKING DOWN STREAM



FRONT ELEVATION
LOOKING TOWARD ARGENTA

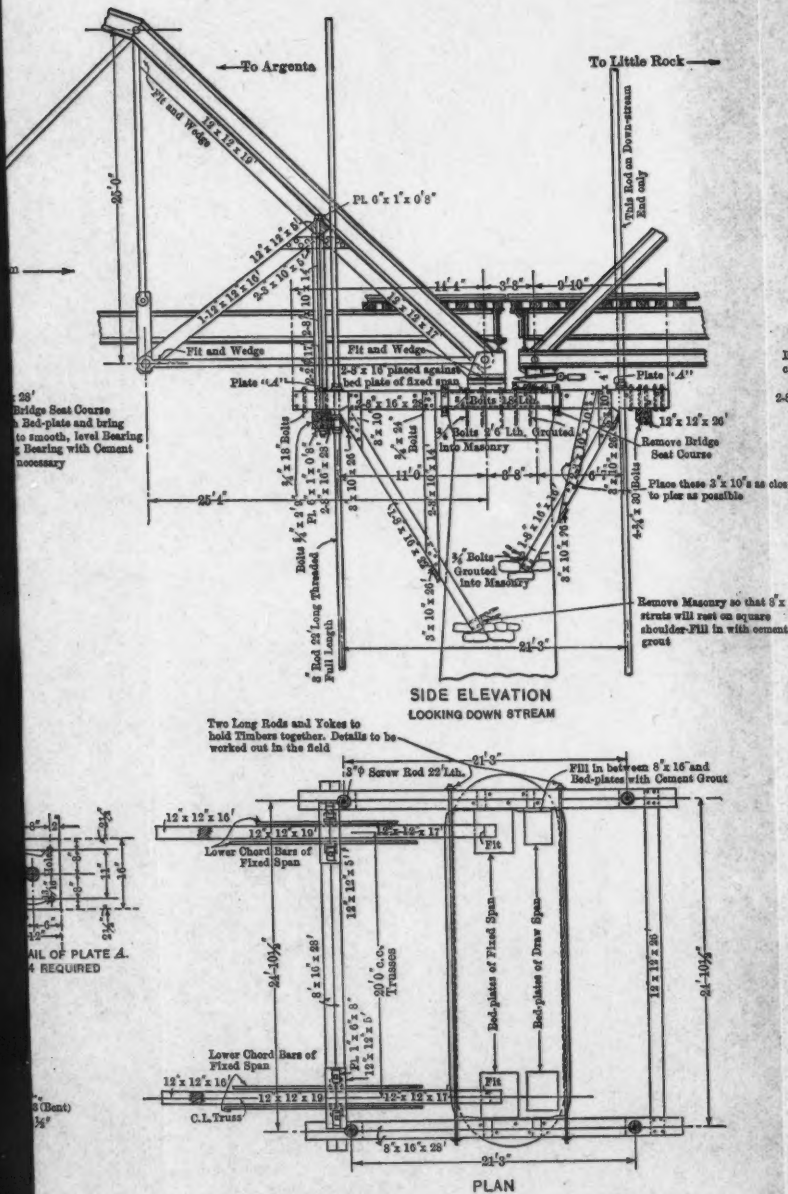


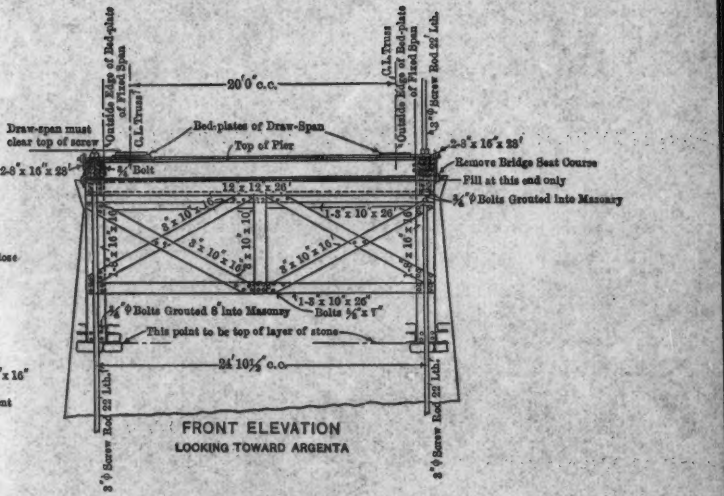
CHAIR DETAILS
4 REQUIRED



PLAN

PLAN SHOWING METHOD OF SUPPORTING AND LOWERING ANNULAR CAISSON USED IN REPAIRING PIER 3 LITTLE ROCK JCT. BRIDGE


To Argenta
To Little Rock
Fit and Wedge
Pl. 0"x 1"x 0'8"
This Rod on Down-stream End only
Fit and Wedge
Fit and Wedge
2-8 x 16 placed against bed plate of fixed span
Plate "A"
Plate "A"
Bridge Seat Course
Bed-plate and bring to smooth, level Bearing
Bearing with Cement necessary
¾" Bolts 2'6" Lth. Grouted into Masonry
12"x 12"x 26'
Remove Bridge Seat Course
Place these 3"x 10"s as clos to pier as possible
¾" Bolts Grouted into Masonry
Remove Masonry so that 3"x struts will rest on square shoulder-Fill in with cement grout
8" Rod 22' Long Threaded Full Length
Bolts ¾" x 2'6" Pl. 0" x 1'-0½" x 2'-4" x 16" x 28"
25'-4"
11'-0"
8'-8"
21'-3"
SIDE ELEVATION
LOOKING DOWN STREAM
Two Long Rods and Yokes to hold Timbers together. Details to be worked out in the field
¾"ϕ Screw Rod 22'Lth.
Fill in between 8"x 16" and Bed-plates with Cement Grout
12"x 12"x 16'
12"x 12"x 19'
12"x 12"x 17'
Fit
Lower Chord Bars of Fixed Span
12"x 12"x 8'½"
8"x 16"x 28'
20'0" c.c. Trusses
Bed-plates of Fixed Span
Bed-plates of Draw Span
12"x 12"x 26'
24'-10½"
21'-10½"
Lower Chord Bars of Fixed Span
C.L. Truss
8"x 16"x 28'
21'-3"
PLAN
AIL OF PLATE A.
4 REQUIRED



FRONT ELEVATION
LOOKING TOWARD ARGENTA

PLAN SHOWING METHOD OF SUPPORTING AND LOWERING ANNULAR CAISSON USED IN REPAIRING PIER 3 LITTLE ROCK JCT. BRIDGE

UNIV. OF MICH.

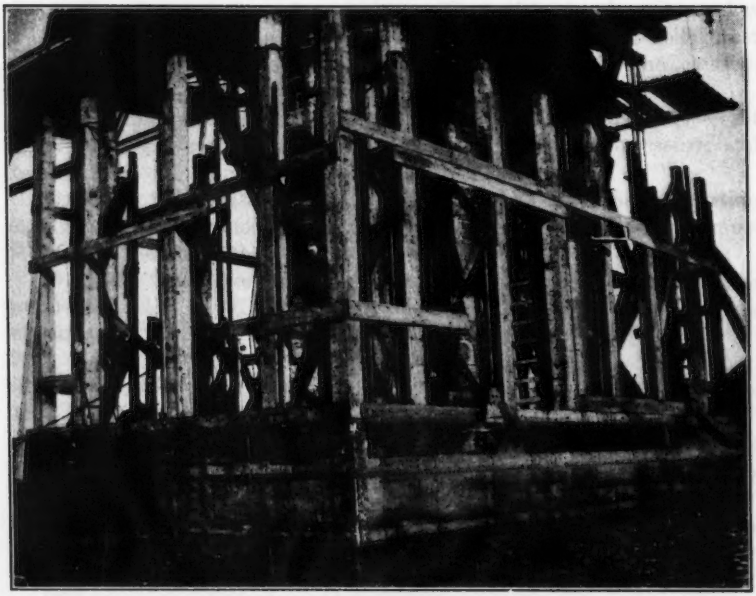


FIG. 25.—LOWERING COFFER-DAM AROUND PIER 3.

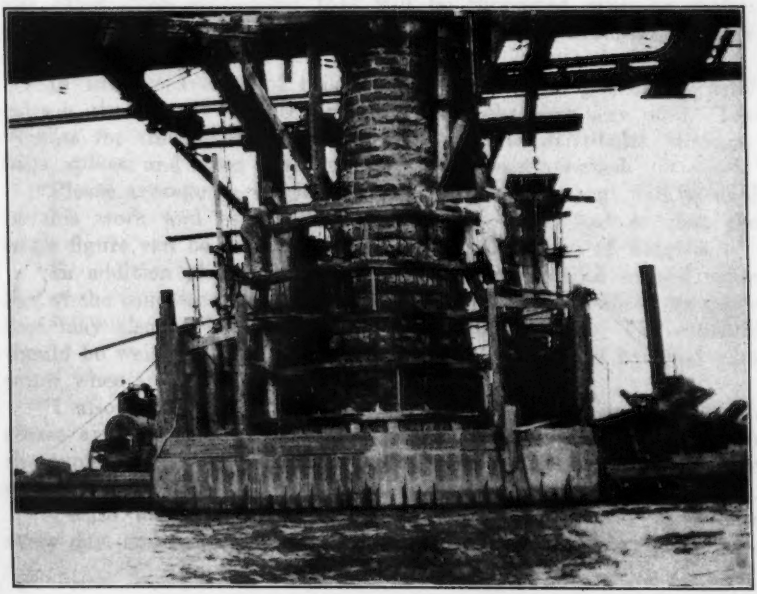
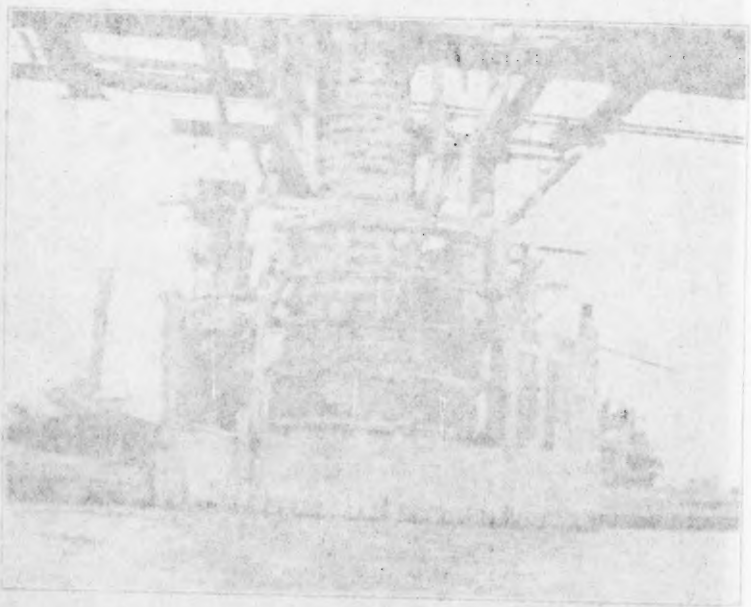


FIG. 26.—STEEL CAISSON AND COFFER-DAM AROUND PIER 3.



"3. In order that the caisson may be filled with air when it reaches the bed of the river, the total weight suspended from the rods must be approximately equal to the weight of water within the caisson, which will be expelled by the application of the air.

"It is expected that when the air is applied the unbalanced weight of the structure will be practically supported by the air pressure, and there will be only a nominal load on the rods.

"4. After the caisson lands on the sand, so that the supporting rods become slack when the nuts are turned, additional weight must be provided to force the caisson into the ground.

"5. After the caisson has reached bed-rock and the working chamber has been sealed by filling it completely full of concrete, there must be sufficient total weight to hold down the structure and prevent it from rising, on account of water pressure caused by the head of water from the surface of the rock to water surface, to be assumed acting over the entire base of the caisson.

"The attached table has been worked out in detail to show the various steps that should be taken in the construction of this caisson and coffer-dam.

"It is probable, of course, that you will not follow the exact procedure outlined, but you should follow it sufficiently close, so that the total weight given in column 'N' will never exceed 325 000 lb.

"Please note that the weights given in the table are assumed weights, and there has been omitted the weight of men, tools, lines, and other small equipment that will be supported upon or within the coffer-dam during sinking; the weight of such material must be added to the total weight.

"In the preparation of this table, the actual weight of the steel caisson, the towers, and the air shafts and locks have been used. The weights for timber, concrete, and iron, such as drift-bolts, through-bolts, spikes, and other small hardware have been assumed.

"Please arrange to weigh sufficient of the timber that will be used in this work and estimate its weight per cubic foot so that the latter figure can be used in your accurate calculation of weights.

"In addition please arrange to make up and weigh several cubic feet of the concrete that will be used, in order that its weight per cubic foot may also be known for use in the calculations. The concrete should be weighed when full of water as it will always be filled with water when submerged.

"I also hand you a number of white prints in blank, on which please arrange to enter notes and figures similar to those shown on the typical summary handed you herewith and arrange to send me one of these weekly with copy direct to Bridge Engineer, C. E. Smith.

"Please understand that this table must be posted up to date every day, and you must instruct your Inspector at the bridge to have

it always in shape for inspection and approval by the Bridge Engineer at all times, as the Bridge Engineer will inspect it at such times as he may visit the bridge.

"Please understand that an increase of the loads over the weights given you above would be very disastrous, as it would undoubtedly result in the breaking of some portion of the supporting structure, and might have a disastrous effect upon Pier 3.

"Please note by reference to the typical summary that the application of air to the caisson will make the structure practically self-supporting, but it is not safe to assume that the structure can be supported by the caisson full of air, as a leak in the caisson or in the supply pipe (which is very long), or a failure in the air compressor, or reservoir, or boilers, or a blow-out under the cutting edge, would immediately decrease the supporting power and increase the load in such a way as to throw a very heavy stress upon the rods, which might result in their failure.

"The caisson must be carefully adjusted level when it is first placed, and must at all times thereafter be kept perfectly level as the lowering of one corner faster than any of the others will result in racking the caisson and coffer-dam to such an extent that heavy leaks may be expected. It will be a very easy matter to keep this structure perfectly level by proper manipulation of the screws.

"In case one corner or one portion of the cutting edge rests upon an obstruction, the obstruction must be removed before further sinking is done. After the caisson reaches a bearing upon the bed of the river and the air has been applied, the structure will, to all intents and purposes, be floating, and it will be necessary to supply additional weight to force it down through the sand. This weight will be provided by placing concrete in the end compartments under water.

"The plans call for these compartments to be supplied with 2-in. forms for concrete for their entire height, so that, if necessary, the entire height of the structure can be built up at the ends to provide this additional weight. In addition it is desired that temporary 2-in. sheeting be placed on the inside of the long sides to prevent sand and other material running out of the old crib on top of the concrete. In no case, however, must any concrete be placed in the long sides, as it is desired that those spaces be left open until after the old crib is repaired.

"It will be impossible to add the weight in the end pockets while the structure is swinging from the rods, as it would not be practicable to provide a support of sufficient strength to support such a very heavy load.

"It is impossible to say how much concrete must be added in these end compartments to force the caisson through the material in

the bed of the river, as that resistance cannot be calculated, but the conditions must be conquered when encountered.

"After the caisson has reached bed-rock and the working chamber has been filled with concrete, the end pockets will be filled with concrete to a height of 25 ft. above the reinforced concrete roof, part of this material being placed during sinking through the sand and the remainder after the caisson has been sealed."

The weight of the steel caisson, the coffer-dam surmounting it, and the layer of concrete over the roof were not sufficient to sink the caisson after the river bed took the load off the rods. Inner forms for concrete in the ends of the coffer-dam were built up as the caisson was lowered, and concrete was deposited in them with bottom-dump buckets as the weight became necessary. Rods were placed in this concrete projecting from the inner face so as to form a bond with the remainder of the concrete to be placed later.

While waiting for the concrete in the working chamber to set, preparatory to pumping out the coffer-dam, a rapid rise came down the river and overtopped the dam on February 29th, 1912. The water subsided in about 6 weeks, and when pumps were started it was found that the coffer-dam had acted as a settling basin and was full nearly to the top with fine packed sand. The removal of this sand proved very burdensome, slow, and expensive, especially on account of the presence of the numerous braces. In the face of another rise, the dam was extended up to 52 ft. above the caisson, but the water went away over that again several times and stayed up for a considerable period, depositing more sand within the dam. The removal of this material, however, presented no difficulty other than the cost and delay. One pump easily controlled the leakage up to a 50-ft. head, giving a dry dam within which to work on the old pier. The filling of the old pier was found to consist of some rip-rap and more sand. In order to remove the sand, much rock had to be taken out. In fact, all the loose rock was removed with the sand, and only the tight rock was permitted to remain. The latter and the entire inside of the crib was washed out with a strong jet and then filled carefully with concrete. On account of the rather rapid settlement of the pier up stream, that end of the crib was first concreted, and, following the placing of that stiffener, the squeezing of the timbers at the down-stream end caused the pier to settle in the other direction, righting itself several inches. The entire space inside the coffer-dam

was filled with concrete, being brought up as the concreting of the crib progressed.

After filling the crib and the surrounding space with concrete, the pier was safe, but, as it was canted so much that it presented a poor appearance, and as the draw-span had only a very small bearing on the pier and was causing the upper courses of masonry to break away, the pier was encased in a reinforced concrete shell. As the new caisson had made the footing large enough, the reinforcement was built for double-track. Plate XXXVII shows the details of the reinforced pier.

The tie-bars at the bottom were placed through bolts cut in the pier, and grouted in. The bridge seats under the spans were entirely renewed in concrete finished at the proper level for double-track spans. The difference in level was made up by concrete pedestals placed in sections, as shown by Fig. 27.

No falsework was used for the support of either span, and the bridge was never out of service during the conduct of the work.

The outer surface of the timbers in the crib under Pier 3 were squeezed down at the ends an almost imperceptible amount. The upper timber was burst open on account of the crushing load.

Fig. 18 is a view of Pier 3 after the completion of the work. The equipment used in this work is listed in Appendix E.

Reconstruction of Pier 4 in 1912.

In anticipation of further work to be done at Pier 4, and in recognition of the fact that it was in much worse condition than Pier 3, it was decided to replace the rods and yokes holding Piers 3 and 4 to the intermediate span by a pair of cables attached in such a manner as to relieve the spans from the pull. As an anchor for each cable, two 4 by 1-in. eye-bars, together with substantial anchors, as shown by Fig. 28, were embedded in the new concrete of Pier 3 near the top just beyond the ends of the old pier.

Before proceeding with work at Pier 4, two 1¾-in. steel cables with cast-steel clevises were attached to the anchor-rods at Pier 3 and suspended from the span. At Pier 4 two 24-in. I-beams were laid on flat, north of the pier, and placed in contact with two 60-ton jacks which were placed horizontally with their bases securely braced against the face of the pier. Tension was put in the cables by the jacks, and the

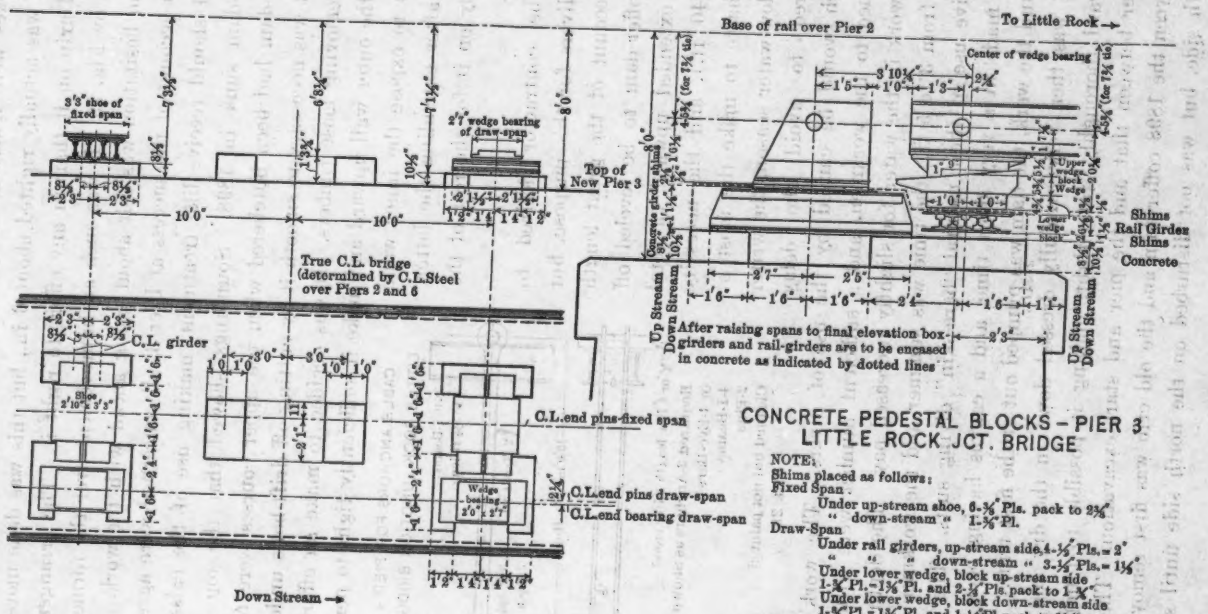


FIG. 27.

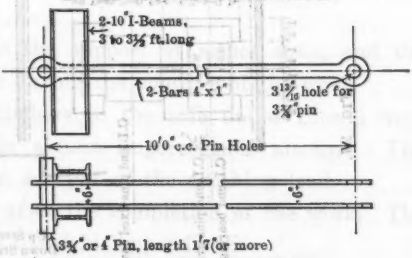
CONCRETE PEDESTAL BLOCKS – PIER 3 LITTLE ROCK JCT. BRIDGE

NOTE:
Shims placed as follows:
Fixed Span
Under up-stream shoe, 6-⅜" Pls. pack to 2⅜"
" down-stream " 1-⅜" Pl.
Draw-Span
Under rail girders, up-stream side, 4-½" Pls. = 2"
" " " down-stream " 3-½" Pls. = 1½"
Under lower wedge, block up-stream side 1-¾" Pl. - 1⅜" Pl. and 2-½" Pls. pack to 1-¼"
Under lower wedge, block down-stream side 1-¾" Pl. - 1⅜" Pl. and 1-½" Pl pack to 1⅛"
Machinery does not drive up wedges to within about 4" of full entrance up-stream side and 6" down-stream side.

old yokes were removed. It was found possible to kick Pier 4 over and it was actually righted about 6 in., but this was done more to get a good grip on it than in any effort to right it. The arrangement is shown by Fig. 29. It was such an improvement over the former yokes that no hesitation was felt about going ahead with the work.

On account of the success at Pier 3, all concerned were agreed that Pier 4 should receive like treatment, making use of the caisson and coffer-dam sunk in 1898. Soundings showed that the top of that coffer-dam had been butchered when the upper courses were removed, and it was necessary to smooth it by cutting off drift-bolts under water and removing loose timbers. It was decided to make an effort to extend the outer wall upward and make it sufficiently tight to permit the pump to expose the inner wall, and, after extending the latter, to rely on it for the rest of the work.

CABLE ANCHORS FOR PIER 4
LITTLE ROCK JUNCTION BRIDGE

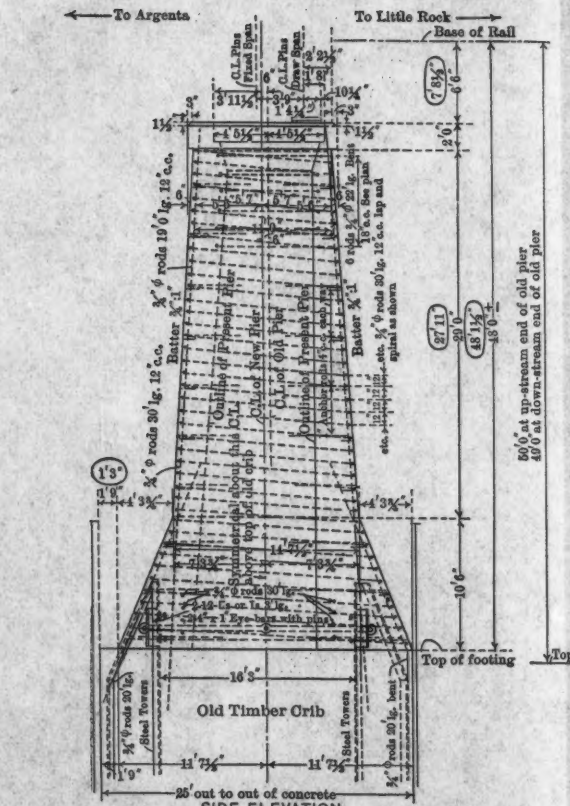


Required 2-Anchors as Shown
or 4-Eye-Bars
4-I-Beams
2-Pins
Cleaned but not painted

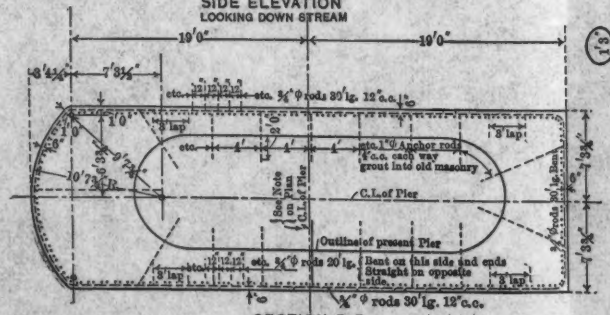
FIG. 28.

The contractor desired to use divers for this purpose, but on account of the great length of coffer-dam to be leveled off and extended upward (a total of 240 ft.), and the necessity for haste to make the most of the low-water season, the writer desired to avoid the delay which would be caused by the use of divers. The work was explained to the workmen, and a sufficient number were willing to do the work in the water for slightly increased pay. The depth of water was from 3 to 5 ft.; the time was summer and the workmen made extensive use of their natural "bare-skin" diving suits. The extension was made in a very short time, and a canvas having been stretched around the work, the dam was pumped out at the first trial. The inner wall was then very carefully dressed down in the dry and extended upward thoroughly water-tight, making it possible to pump out the water between that and the pier and start excavation. The concrete between the 1898 coffer-dam and the old crib was first removed on the south side, but was not disturbed on the north side until some time

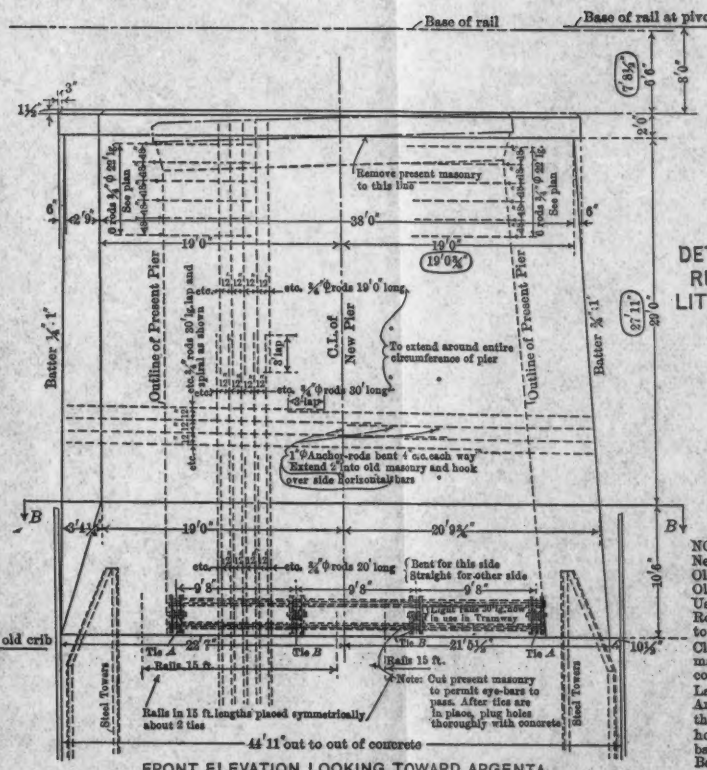
DETAILED MASONRY PLAN FOR RECONSTRUCTION OF PIER 3 LITTLE ROCK JUNCTION BRIDGE



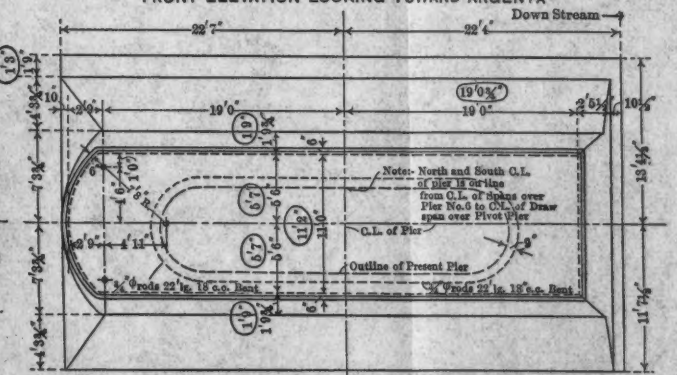
SIDE ELEVATION
LOOKING DOWN STREAM



SECTION B-B



FRONT ELEVATION LOOKING TOWARD ARGENTA



PLAN

NOTE:-
New work shown thus ——
Old work to be removed shown thus - - -
Old work to remain in place " "
Use 1:2:4 Concrete
Rods not to come nearer than 3" to face of concrete
Clean and moisten thoroughly old masonry where joined by new concrete.
Lap all rods 3'0"
Anchor-rods, 1" Rd., must be grouted thoroughly into old masonry and hooked over outside horizontal bars thus ⊃
Bevel 1" all corners 90° or less

Figures enclosed thus (7'8½") show pier as built. Top of old pier projects above top of new pier 4" at up-stream end and 9" at down-stream end.

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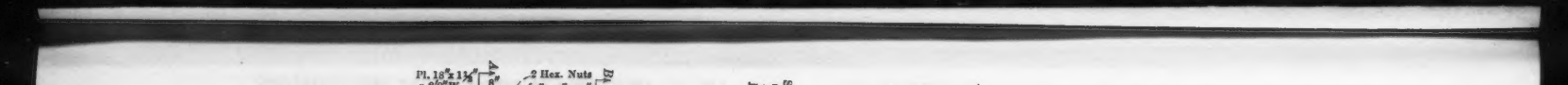

Pl. 16" x 1¼"
2 Hex. Nuts

later, after the yokes and cables were in place anchoring the pier to Pier 3.

The removal of the material down to rock was slow and tedious. The old coffer-dam had been badly racked during sinking, and large leaks were found all the way down. They were overcome, and the rock was laid bare and covered with concrete up to the top of the caisson of the old pier, after which the crib was cleaned out, filled with concrete, and surrounded as at Pier 3.

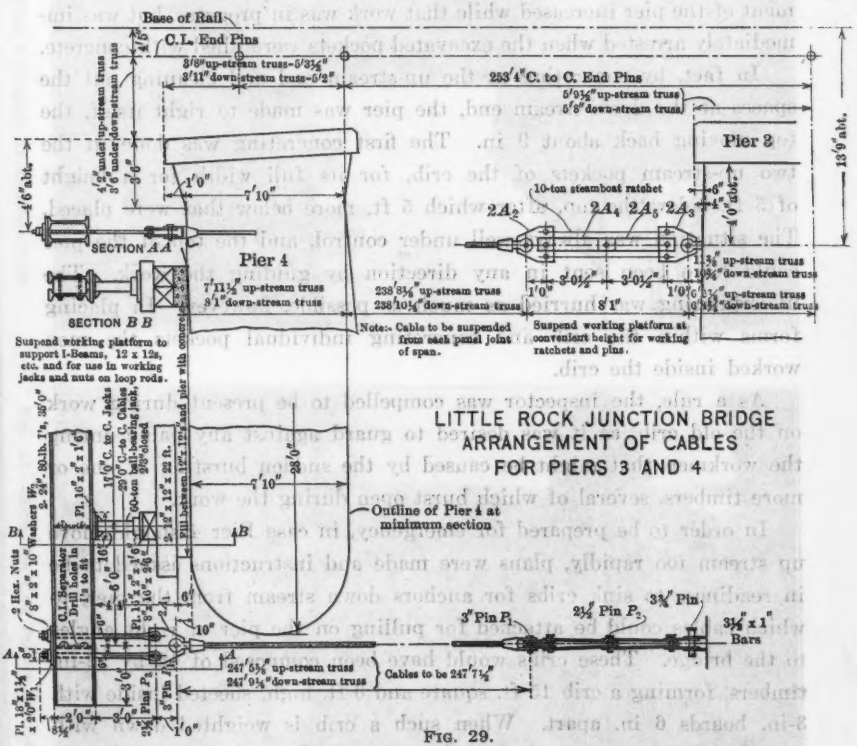


FIG. 29.

There was always a great flow of water from the south side, and in excavating down it was found that it came through under the cutting edges of the 1898 coffer-dam where a dip in the rock had been bridged over and was supposed to have been filled with concrete, but apparently was not. Several bad blows, carrying many yards of sand and flooding the excavation, occurred under the 1898 caisson, and it

was not possible to pump down to the bottom until after the material had been removed and the leak stopped by placing a sealing course of concrete under water.

Some of the timbers were crushed to 6 in. in height, and on account of the inclination of the pier new cracks developed in the timbers while the excavation progressed. The weakest portions of this crib were concreted as they were reached. Attention was given to the up-stream end, and excavation was first made there. The movement of the pier increased while that work was in progress, but was immediately arrested when the excavated pockets were filled with concrete.

In fact, by concreting at the up-stream end and cleaning out the spaces at the down-stream end, the pier was made to right itself, the top moving back about 9 in. The first concreting was done in the two up-stream pockets of the crib, for its full width for a height of 5 ft. below the top, after which 5 ft. more below that were placed. The situation was always well under control, and the top of the pier could have been sent in any direction by guiding the work. The underpinning was hurried as much as possible, however. In placing forms within the crib and concreting individual pockets the men worked inside the crib.

As a rule, the inspector was compelled to be present during work on the old crib, as it was desired to guard against any panic among the workmen that might be caused by the sudden bursting of one or more timbers, several of which burst open during the work.

In order to be prepared for emergency, in case Pier 4 should move up stream too rapidly, plans were made and instructions issued to be in readiness to sink cribs for anchors down stream from the pier, to which cables could be attached for pulling on the pier at right angles to the bridge. These cribs would have been composed of 12 by 12-in. timbers, forming a crib 15 ft. square and 6 ft. high, sheeted inside with 3-in. boards 6 in. apart. When such a crib is weighted down with rock and dropped on the bed of the Arkansas River it becomes buried immediately and forms an almost immovable anchor. It was not found necessary to sink such cribs, however.

No falsework was used while Pier 4 was under reconstruction; the pier carried traffic at all times.

Figs. 30 and 31 are two views of Pier 4 during the reconstruction. The old timber yoke and rods connecting to the trusses, placed in

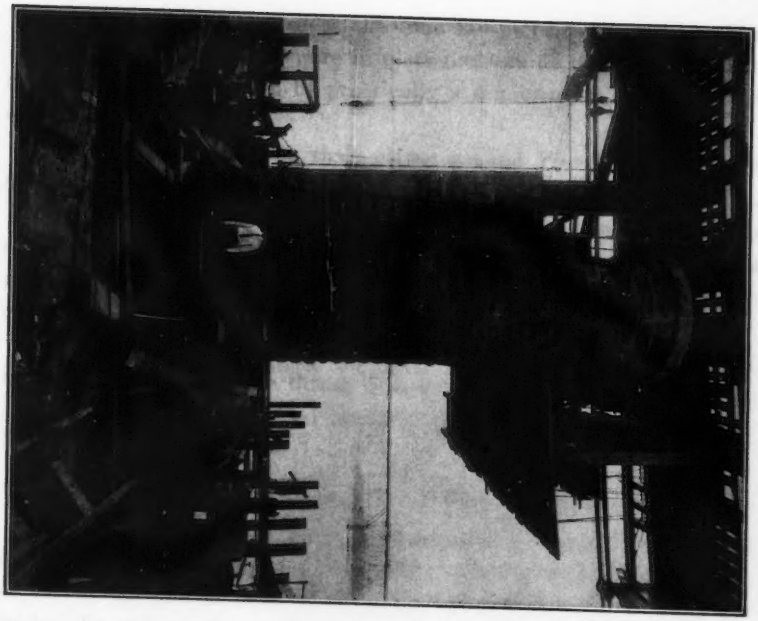


FIG. 30.—END VIEW OF PIER 4 DURING RECONSTRUCTION.

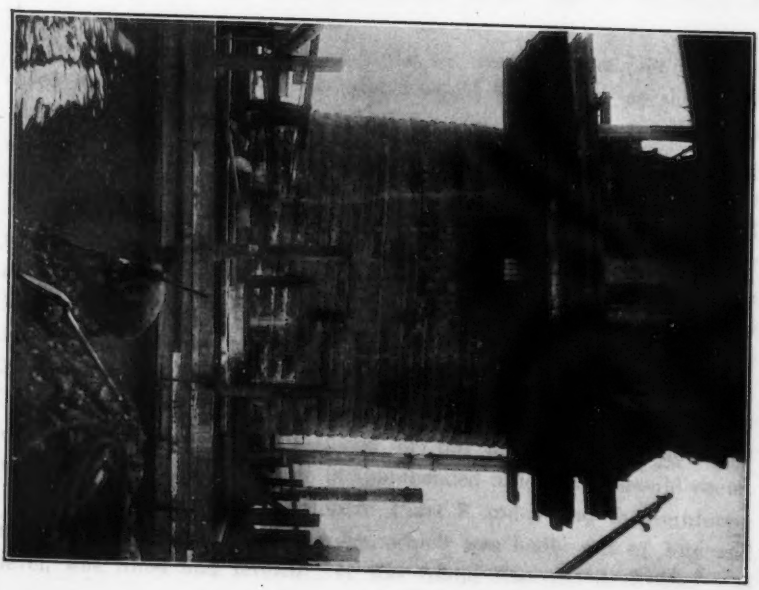


FIG. 31.—SIDE VIEW OF PIER 4 DURING RECONSTRUCTION.

1908, had not been removed at the time the picture was taken. The I-beams and 1¾-in. cables were in place and can be readily seen. The completed pier appears to be practically a duplicate of Pier 3, shown on Fig. 18.

The reinforced concrete shell placed around Pier 4 is practically a duplicate of that placed at Pier 3. It is shown on Plate XXXVIII. The tie-bars at the bottom were passed through the pockets in the crib and concreted in, instead of being passed through the pier. At this pier the ends of the spans were shifted sideways to permit the construction of each concrete pedestal in one piece.

As Pier 5 had moved north somewhat in the past, as a result of its own weakness and the thrust from Pier 4, anchors were buried in Pier 4 and the cables shifted ahead, Pier 4 now acting as an anchor for Pier 5.

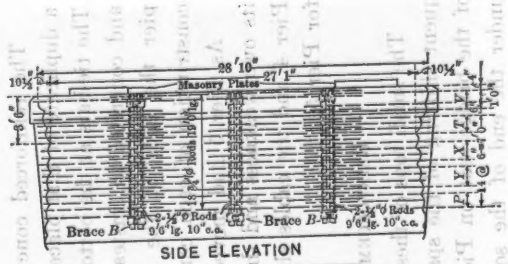
Reinforcement of Top of Pier 5 in 1913.

The small tops of these piers, their reported movement, and the frequent shifting of the spans, all contributed to a very poor condition of the bridge seat on Pier 5, on which the stones were breaking out under the end of the south span which rested close to the edge of the pier.

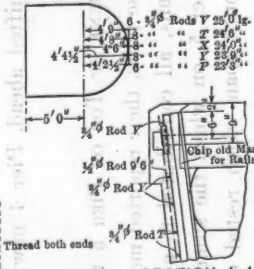
To strengthen the bridge seat, the top 10 ft. of the pier was encased in a reinforced concrete jacket about 9-in. thick, as shown by Fig. 32. The sides were connected by passing the reinforcing rods around the curved ends of the pier and also by three sets of rails set vertically in the reinforced concrete at the middle and quarter points of the length of the pier. The lower ends of these rails were attached to the pier by 2-in. anchor-bolts set in grouted holes. Their upper ends were connected by 2-in. rods extending across the pier tops in channels cut in the bridge seat and filled with concrete. All cracks in the pier top were carefully grouted. The bridge seat appears to have taken on a new lease of life.

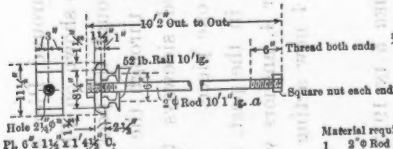
It is expected that, as Pier 5 is quite a distance from the ordinary channel, there is little likelihood of the material surrounding it washing away, and no trouble is apprehended unless that should occur.

Following the reconstruction of Piers 3 and 4 and the reinforcement of Pier 5, the entire bridge, which was badly out of line and level, was lined and leveled. It was found that the draw-span was



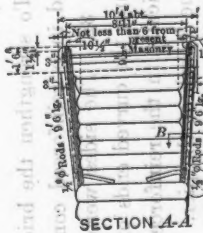
Bill of Reinforcing Bars			
36	¾"ϕ Rods - 19'0" long		
6	" " 25'0" "		V
8	" " 24'6" "		T
8	" " 24'0" "		X
8	" " 23'9" "		Y
6	" " 23'3" "		P
12	½"ϕ " 9'6" "		
	Use Deformed Rods		
60	Pcs. No. 16 Wire 2'6" long		

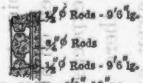




Material required for 1 Brace- *B*

1	2"ϕ Rod 10'2"lg.	Mark *a*
2	" " 3'0" "	Bent Mark *b*
4	6"x 1½"Pls. 1'4½"	Mark *c*
4	Square Nuts for 2"ϕ Rods	
4	Pcs.- 52-lb. Rails 10'0"lg.	





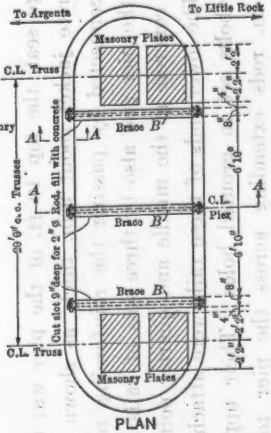
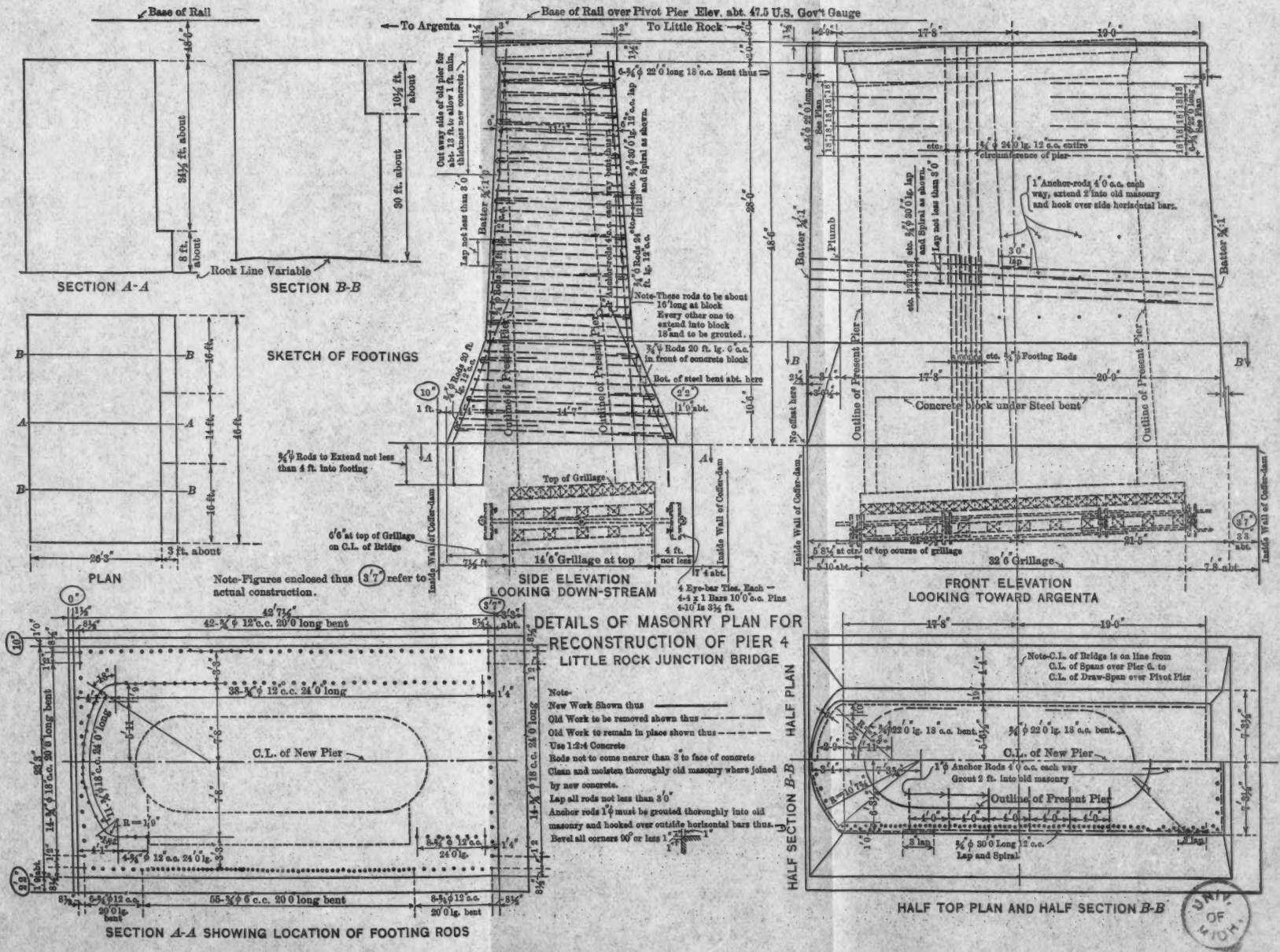


FIG. 32.


Base of Rail
Rock Line Variable
SECTION A-A
SECTION B-B
SKETCH OF FOOTINGS
PLAN
Note-Figures enclosed thus (3'7) refer to actual construction.
Base of Rail over Pivot Pier Elev. abt. 47.5 U.S. Gov't Gauge
To Argenta
To Little Rock
Outline of Present Pier
Top of Grillage
Grillage at top
Inside Wall of Coffer-dam
SIDE ELEVATION
LOOKING DOWN-STREAM
Concrete block under Steel bent
Outline of Present Pier
Grillage
FRONT ELEVATION
LOOKING TOWARD ARGENTA
DETAILS OF MASONRY PLAN FOR
RECONSTRUCTION OF PIER 4
LITTLE ROCK JUNCTION BRIDGE
Note-
New Work Shown thus
Old Work to be removed shown thus
Old Work to remain in place shown thus
Use 1:2:4 Concrete
Rods not to come nearer than 3" to face of concrete
Clean and moisten thoroughly old masonry where joined by new concrete.
Lap all rods not less than 3'0"
Anchor rods 1" must be grouted thoroughly into old masonry and hooked over outside horizontal bars thus
Bevel all corners 90° or less
C.L. of New Pier
Outline of Present Pier
HALF PLAN
HALF SECTION B-B
HALF TOP PLAN AND HALF SECTION B-B
SECTION A-A SHOWING LOCATION OF FOOTING RODS

somewhat twisted and one end hung lower than the other. As far as could be done without injury to the span, the inequalities were corrected.

The work on the three piers and spans cost about $100 000, which is much more than it would have cost had the work on Pier 3 been started earlier and the floods avoided.

The reinforcement of Piers 3 and 4 was executed on a force-account basis by Bates and Rogers Construction Company. The adjustment of the spans and the reinforcement of Pier 5 were executed by the Railway Company forces.

Instrumental Observations.

Various systems of observations had been used to get a record of the extent and character of movement, but they had not been closely followed up, so that the old records of movement are intermittent. No figures are available previous to 1905, and measurements prior to 1908 were not strictly comparative because the errors of observation were often greater than the movement, but over any extended period they indicated the general trend of the movement. In March, 1908, triangulation monuments were placed on Pier 4 and on the river banks for monthly observations. The movement of the piers and spans, however, was such that it was necessary to change the points continually in order to get a clear view through the bridge, and as that rendered the observations of questionable value, this method was abandoned in 1910 for that in use at the present time. The character of the observations taken since that time is best described by the following copy of the instructions issued to observers on July 15th, 1910:

"Instructions for Taking Observations on Piers of Little Rock Junction Bridge, Little Rock, Arkansas.

"St. Louis, Mo., July 15th, 1910.

"Character of Observations.

"The observations consist of four independent determinations, as follows:

"*Line.*—Determining the location of two points on each pier by line relative to fixed monuments on shore.

"*Distance.*—Determining the location of said points by measurements from fixed monuments on shore.

"*Grade.*—Determining the elevation of said points with reference to established bench-marks.

"*Soundings.*—Determining profile of river bed entirely across the river at the center line of track and at the up-stream and down-stream ends of the piers.

"General Remarks.

"The observer must provide himself with a complete copy of these instructions and accompanying drawings, and study them carefully before making the observations.

"The two points on each of the piers consist of copper plugs, ¾ in. in diameter, set in the masonry, located and placed as shown on Sheets 1 and 4. The fixed monuments on shore consist of short sections of old rails set in the ground, located and placed as shown on Sheets 1, 2, 3, and 4.

"The points on the piers and the monuments on the Argenta side of the river being several feet below the level of the track, it is necessary to use a plumb-bob over each point. In order to protect the plumb-bob and string from atmospheric disturbances, a box, open at both ends and cut to proper length, as shown on Sheets 4 and 5, has been provided for each point. The boxes for Monuments *I* and *K* are fixed in position at the top end by hinges screwed to the under side of the bridge. This enables the boxes to be swung aside when desirable.

"The boxes for the points on the piers are marked *x*, *y*, etc., to correspond with the point over which they should be used.

"The boxes are held in place by cleats connecting their tops to the ends of the bridge ties. Those on the west or up-stream end of the piers extend 6 in. above the top of the floor-beams, those on the east or down-stream ends of the piers extend up to the under side of the plank walk. The boxes are provided with adjustable cross-arms for supporting the plumb line. The cross-arms for the west boxes may be tacked directly to their top; those for the east boxes may be tacked to blocking supported on the plank walk.

"Points are located on these adjustable cross-arms directly over the points on the piers by means of a plumb-bob suspended through the box directly over the respective points on the piers.

"Monuments *B*, *C*, *I*, and *K* are to be used for establishing the reference lines.

"The direction and character of the wind, the nature of the weather, distance base of rail to water surface, temperature, and any other conditions affecting the observations must be recorded on each occasion.

"Method of Procedure for Line.

"Set the transit over Monument *C* and check line *A B C D*; then set it over *H* and check lines *G H I* and *H J K L*; set over *M*

and check lines $M\ J\ I$ and $N\ M\ K$. If these lines check, the monuments may be assumed to be correct. With transit at M and foresight on I the location of x_7 may be checked.

"Next move transit up to deck of bridge and set it over point K, sighting on C and establishing line at points, y_7, y_6, y_5, and y_4; then over point I, sighting on B and establishing points, x_6, x_5, x_4, and x_3.

"Next move transit to other end of bridge and set it over C, sighting on K and establishing points, y_1, y_2, y_3, and y_4.

"The distances between the points thus established and the original points formerly established on the copper plugs measured on the cross-arms, at right angles to the line of sight, must be recorded, as these distances indicate the movement of the piers in an east and west direction, at right angles to the direction of the bridge.

"This observation must be taken at least twice, the observers changing places and repeating all operations until their observations check.

"Method of Procedure for Distances.

"The measurements from fixed point on shore are made along lines $B\ I$ and $C\ X$, points B and C being taken at Station 0, and the station of each of the points, x, y, etc., as well as I and K being recorded.

"On line $C\ K$ the tape is permitted to rest on the tops of the floor-beams; on line $B\ I$ (which follows the footwalk) wood blocks about 4 in. high must be placed over each floor-beam to reproduce the conditions of support obtained on line $C\ K$ and to eliminate the effect of the temperature of the walk on the tape.

"The distances are measured with a 200-ft. tape stretched to a tension indicated as 15 lb. on a spring balance. Monuments S_1 and S_2 have been carefully set 200 ft. apart for standardizing the tape, which is correct at 62° Fahr. For other temperatures the measured distances must be increased (+) or decreased (—) by the amounts shown in the following table.

"Average Temperature.

Meas.	12	22	32	45	52	62	72	82	92	102	112
Bx	— .057	— .045	— .034	— .023	— .011	0	.011	.023	.034	.045	.057
Bx	— .112	— .089	— .067	— .044	— .022	0	.022	.044	.067	.089	.112
Bx	— .174	— .139	— .104	— .069	— .035	0	.035	.069	.104	.139	.174
Bx	— .258	— .206	— .155	— .103	— .052	0	.052	.103	.155	.206	.258
Bx	— .340	— .271	— .203	— .135	— .068	0	.068	.135	.203	.271	.340
Bx	— .422	— .338	— .253	— .168	— .084	0	.084	.168	.253	.338	.422
Bx	— .466	— .373	— .280	— .186	— .093	0	.093	.186	.280	.373	.466
Cy	— .053	— .043	— .032	— .021	— .011	0	.011	.021	.032	.043	.053
Cy	— .109	— .087	— .065	— .043	— .022	0	.022	.043	.065	.087	.109
Cy	— .170	— .136	— .092	— .068	— .034	0	.034	.068	.092	.136	.170
Cy	— .254	— .203	— .152	— .101	— .051	0	.051	.101	.152	.203	.254
Cy	— .338	— .270	— .203	— .135	— .068	0	.068	.135	.203	.270	.338
Cy	— .418	— .335	— .251	— .167	— .083	0	.083	.167	.251	.335	.418
Cy	— .461	— .368	— .276	— .184	— .092	0	.092	.184	.276	.368	.461

"Method of Procedure for Grades.

"The top of the steel rail in Monument *C* is the bench-mark elevation. For purposes of checking, any other monument may be used, the elevations being shown on Sheets 2 and 3.

"Set the level about half way between *C* and the south abutment, use point *y* for turning point. Move level to middle of first span and use point *y* for turning point. Continue across all the spans and back in this manner, checking on bench-mark *C* and taking readings on all points on piers in each direction for a check.

"In case the levels check upon returning to *C* and the two readings on each point check, this observation need not be repeated. Otherwise it must be repeated until a check is obtained.

"Method of Procedure for Soundings.

"Soundings shall be taken along the lines $P\ P_1$; $P_2\ P_3$, and $P_4\ P_5$, shown on Sheet 6, preferably by use of a lead line from a boat, except where the ground line is above water, where it can be taken by a weighted tape from the bridge floor. The distance from base of rail to water surface shall always be recorded.

"Soundings shall be taken close to the foundation under each pier, but far enough out to miss it, and at every second floor-beam of each span (about 50 ft. apart).

"This observation for soundings need be taken only once, there being no necessity for its repetition."

Fig. 33 shows the location of the base lines, points on pier, transit points, and other monuments. No movement has been detected in the pivot pier since the repairs in 1898; no movement has ever been detected in Piers 1, 6, and 7.

Fig. 34 shows the movement curve for Pier 3; practically no movement was detected up to 1908. In that year, following the application of the rods and yokes to the ends of the span resting on Piers 3 and 4, Pier 3 moved north (parallel to the bridge) about 10 in. from 1908 to 1913, during which period the reconstruction was carried out, since which work the pier has been stationary. No movement east or west (at right angles to the bridge) was detected in Pier 3 until the summer of 1910, when, for no apparent reason other than the deep scour of the river around the pier, it started up stream at the rather rapid rate of 9 in. in 14 months, the up-stream end settling several inches. This movement brought about the reconstruction of the pier in 1911 and 1912, during which it was found possible, by controlling the underpinning, to make the pier partly right itself, the curve showing that the top moved back (east) about 6 in. during the work.

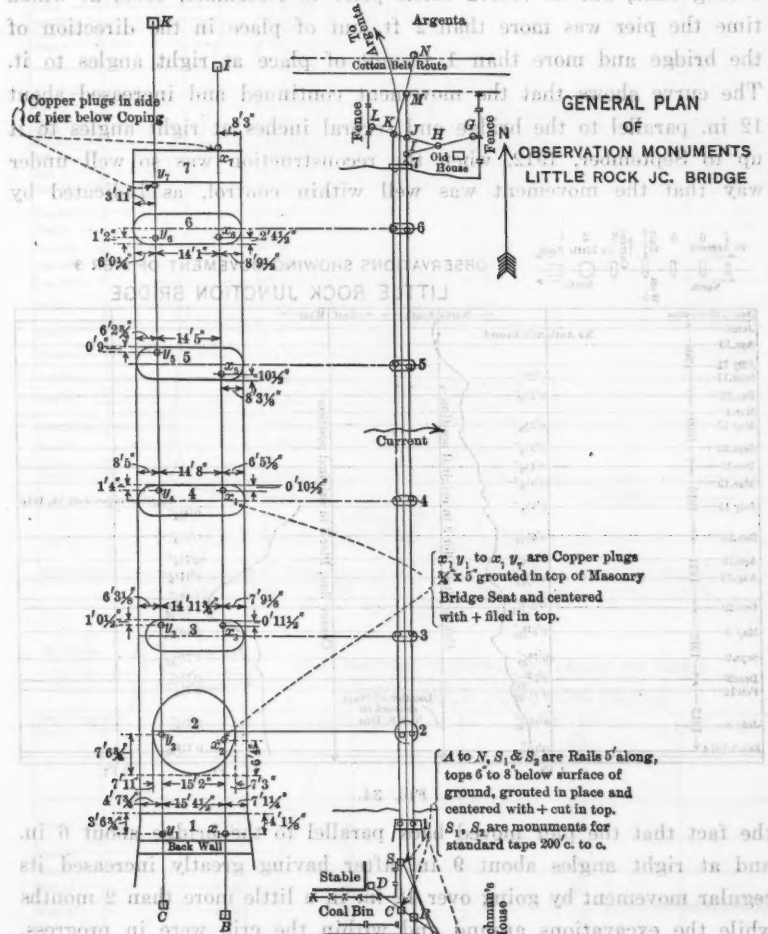

GENERAL PLAN
OF
OBSERVATION MONUMENTS
LITTLE ROCK JC. BRIDGE
Argenta
To
Argenta
Cotton Belt Route
Copper plugs in side of pier below Coping
Fence
Fence
Old House
Current
x_1 y_1 to x_7 y_7 are Copper plugs ¾" x 5" grouted in top of Masonry Bridge Seat and centered with + filed in top.
A to N, S_1 & S_2 are Rails 5' along, tops 6" to 8" below surface of ground, grouted in place and centered with + cut in top.
S_1, S_2 are monuments for standard tape 200'. c. to c.
Back Wall
Stable
Coal Bin
Watchman's House
Telegraph Pole
To Little Rock
St. L. I. M. & S. R.R.
To McGehee
Little Rock

FIG. 33.

Fig. 35 shows the movement curve for Pier 4. The position of the pier in 1898 made it evident that the movement had taken place for a long time, but no record exists prior to December, 1905, at which time the pier was more than 2 ft. out of place in the direction of the bridge and more than 1 ft. out of place at right angles to it. The curve shows that the movement continued and increased about 12 in. parallel to the bridge and several inches at right angles to it up to September, 1912, when the reconstruction was so well under way that the movement was well within control, as indicated by

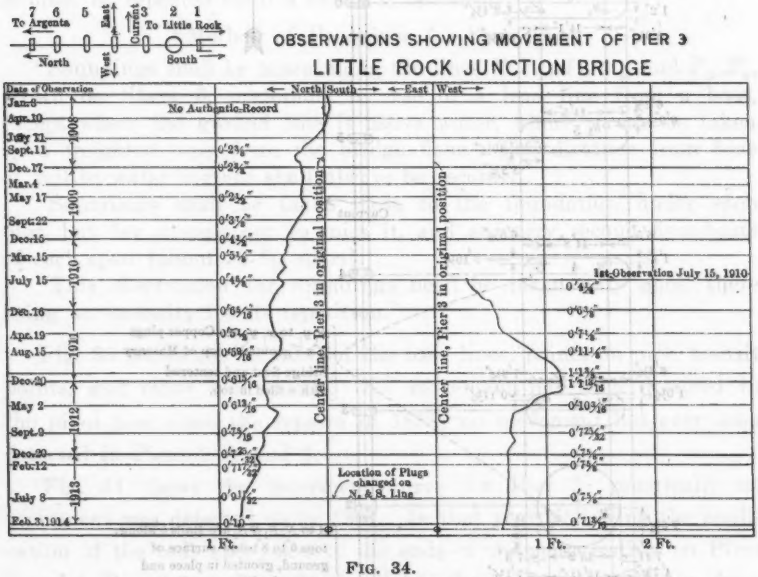


FIG. 34.

the fact that the pier moved back parallel to the bridge about 6 in. and at right angles about 9 in. after having greatly increased its regular movement by going over 12 in. in a little more than 2 months while the excavations around and within the crib were in progress. Since the reconstruction this pier has been stationary.

Fig. 36 shows the movement curve for Pier 5. No movement has ever been detected at right angles to the bridge, but there has been intermittent movement parallel with it. No movement has been detected since the application of the cables anchoring this pier to Pier 5, and no further movement is expected.

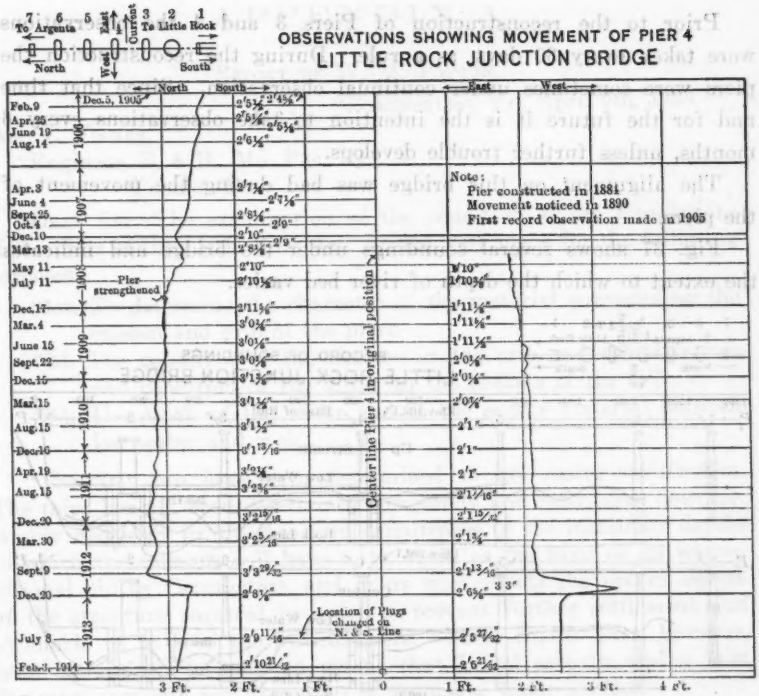

OBSERVATIONS SHOWING MOVEMENT OF PIER 4
LITTLE ROCK JUNCTION BRIDGE
To Argenta
To Little Rock
Current
North
South
West
East
Note:
Pier constructed in 1881
Movement noticed in 1890
First record observation made in 1905
Pier strengthened
Center line Pier 4 in original position
Location of Plugs changed on N. & S. Line
3 Ft.
2 Ft.
1 Ft.
0
1 Ft.
2 Ft.
3 Ft.
4 Ft.

FIG. 35.

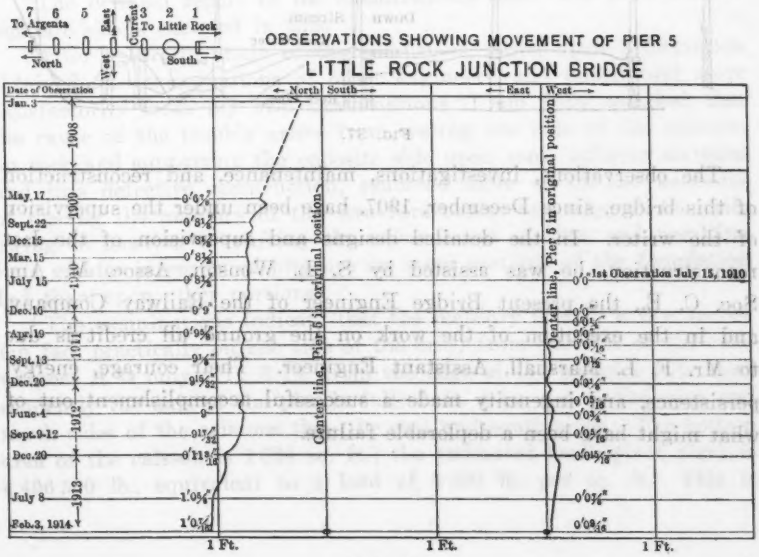

OBSERVATIONS SHOWING MOVEMENT OF PIER 5
LITTLE ROCK JUNCTION BRIDGE
To Argenta
To Little Rock
Current
North
South
East
West
Center line, Pier 5 in original position
1st Observation July 15, 1910
1 Ft.
1 Ft.
1 Ft.

FIG. 36.

Prior to the reconstruction of Piers 3 and 4 the observations were taken every 30 days, as a rule. During the reconstruction the piers were sometimes under continual observation. Since that time and for the future it is the intention to take observations every 6 months, unless further trouble develops.

The alignment on this bridge was bad during the movement of the piers.

Fig. 37 shows several soundings under the bridge and indicates the extent to which the depth of river bed varies.

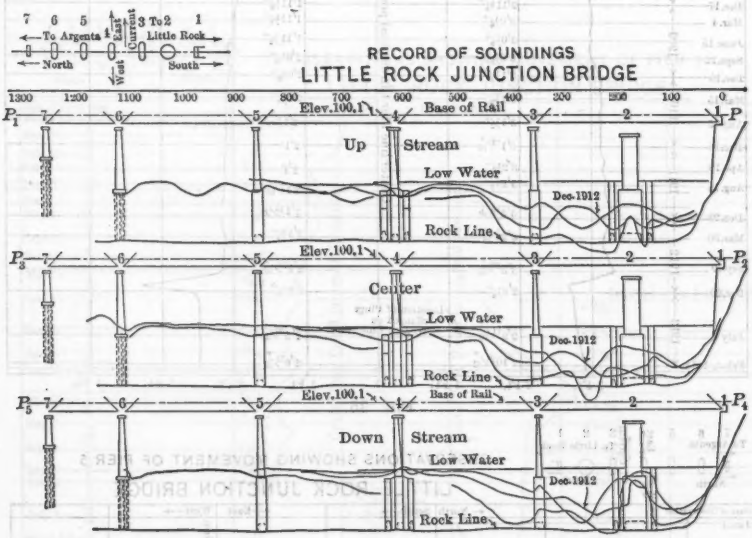


FIG. 37.

The observations, investigations, maintenance, and reconstruction of this bridge, since December, 1907, have been under the supervision of the writer. In the detailed designs and supervision of the last reconstruction, he was assisted by S. L. Wonson, Assoc. M. Am. Soc. C. E., the present Bridge Engineer of the Railway Company, and in the execution of the work on the ground all credit is due to Mr. F. L. Marshall, Assistant Engineer. Their courage, energy, persistence, and ingenuity made a successful accomplishment out of what might have been a deplorable failure.

APPENDIX A

REPORT OF W. M. PATTON.

"BLACKSBURG, VA., April 9th, 1898.

"MR. E. FISHER,
"Engineer, B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"DEAR SIR: The examination of the condition of the foundations of the piers of the Little Rock Jct. Bridge were made for the following purposes:

"1st. To determine the character of the material surrounding the caisson and crib of the piers.

"2d. The condition of the timbers in the crib and the character and condition of the filling in the pockets of the crib.

"3d. The depth of the water and of the earthy material between low water and rock.

"The first two have been determined to my entire satisfaction. The third has not been determined completely and fully, this, however, is only necessary in order to give definiteness to the maximum depths to which the caissons will have to be sunk, as the basis of an unconditional bid by Contractors, and in no wise affects the proper design of the structure required in order to prevent further settlement and to enable us to level the present piers. These depths have, however, been determined to such an extent that I feel safe in saying that the maximum will not exceed 40 ft. from low water surface.

"The detailed report of the examinations made will be found on separate sheets enclosed in this.

"After a careful study of the conditions, based upon information obtained, the examinations of other engineers, and finally and more satisfactorily from my own examinations, I am fully satisfied that the cause of the trouble arises from resting one side of the caissons on rock and supporting the opposite side upon some inferior material and this defective construction, rendered more so by the necessity of placing the masonry of the structure nearer one edge of the crib, owing to locating or sinking the caissons out of line, and resulting in a greater intensity of pressure on some portions of the foundation bed than upon other portions.

"The examinations indicate that the masonry is only a few inches (in fact practically on one side of the crib) from one side of the crib, whereas it is only 2 ft. 8 in. from the edge of the opposite side, consequently the axis of the pier is fully 1 ft. 4 in. nearer the west and north sides of the caissons than the east and south sides. The bottom area of the caisson is 1 024 sq. ft.; the estimated load (pivot pier) is 4 496 500 lb., equivalent to a load of 4 400 lb. per sq. ft. This is

nearly, if not quite, the maximum allowable unit pressure. The eccentricity of the load gives as the maximum pressure over 4 300 lb. pressure per sq. ft., which is in excess of the allowable load upon anything but solid rock. This condition is more noticeable at Pier 4 than at Pivot Pier. At Pier 4 the uniform pressure per square foot of foundation bed is 6 200 lb., and owing to the eccentric loading the maximum pressure per square foot must be over 7 000 lb., a unit load far beyond anything permissible.

"These conditions fully account for the settling of the piers, here it may be asked, why has the settlement been toward the west and not toward the north, if at both of these points the rock is at a greater depth than at the south and east corners? It may be answered that:

"1st. The filling between the bottom of the caisson and the rock at the north corner may be better than at the west corner.

"2d. The west corner, being the up-stream corner, is scoured out to a greater depth than the north corner, thereby reducing to that extent the supporting power of the material.

"The depth of water at the east corner was found to be 16 ft. 9 in., at west corner 22 ft. 5 in. and this, notwithstanding the large amount of rip-rap deposited at and on either side of west corner, and, moreover, other conditions might exist as to why this settlement to the west occurred, rather than toward the north, even assuming that the elevation of the rock at the west corner is the same or even higher than the north corner. We are, however, dealing with fact not a theory, and whatever may be the difference of opinion as to the cause of the trouble, the important question is, what is the proper and best remedy, under known, not probable conditions. Clearly any design with this object in view should enable us to solidify the present material under the westerly half of the caisson, or remove that material and substitute some more solid material for it:

"1. To solidify the material at present between the bottom of the caisson and the rock. The only practicable and available method is known as the Harris' process, which consists in injecting pure (liquid) cement under pressure into the material, thereby converting it into concrete in place. This method while perhaps never employed in a case exactly similar to the one under consideration, has proved to be effective under other conditions and where the original material is clean sand, I am satisfied it is a safe and effective method. The examination showed a remarkably clean sand and gravel from the bed of the river to rock; at the east corner a thin layer of clay and silt, at about 7 ft. above the rock; but as it would only be necessary to apply, at any rate for the present, the process to that half of the caisson from the south through the west to the north, the presence of this clay at the east corner would be of no consequence, and its

distance from the rock would render it not worthy of consideration at all. The conditions are, therefore, exceedingly favorable for the application of the Harris' process. It will be a, comparatively speaking, rather inexpensive process, even recognizing it as patented, probably $10 000 would cover the entire cost. I do not, however, recommend this method, owing to the uncertainty that would necessarily exist as to the results obtained, which could only be determined by time, and in addition it would preclude the possibility of raising the pier to its proper vertical position. If you see proper you can correspond upon the subject with the Harris Company of New York.

"2. To remove the present material under the caisson and substitute it with concrete; the available methods to accomplish this are:

"1st. The Poetsch-Sooysmith freezing process.

"2d. The ordinary coffer-dam.

"3d. The pneumatic caisson, using the old caisson and crib for one of its walls.

"4th. An independent caisson and coffer-dam constructed entirely or partly around the old caisson and crib.

"1st. An ordinary coffer-dam let a few feet into the bed of the river and freezing a wall of the material below to the rock. This method can be relied upon as effective, and the only consideration is in respect to the cost. You can correspond with Wm. Sooy-Smith of Chicago or Charles Sooysmith of New York.

"2d. The ordinary coffer-dam is probably hardly worth considering, as it would be extremely difficult to make a water-tight dam in the kind of material around these piers and of the depth required. There would be many risks to incur of failure and the saving in cost, if any, would hardly justify the employment of such an uncertain method.

"3d. The method that at first seemed to me the best, surest, and most economical was to use a pneumatic caisson, making the old crib and caisson one of its walls. My own examination of the crib and its filling, which entirely corresponds with the report of the divers, discloses the fact that the crib is filled with small rip-rap, sand, and gravel, the latter in all probability was washed in subsequently to the completion of the structure, through the open sides and pockets. This condition convinced me that at least it would be a very risky method, and the risk is so great that I do not consider it justified by the saving in cost, if it should prove successful.

"4th. I have, therefore, been compelled to recommend the adoption of an entirely independent caisson, entirely or partly surrounding the old crib and caisson, leaving a working space of 6 ft. between the old and new caisson. The working designs for this structure with bill of material, estimate of cost and full detail, are sent by express to your address.

"While this design in many of its main features is similar to that designed for many piers constructed by me, there are some novel features introduced which are justified and demanded by the novel conditions, to which it is adapted. The design commends itself for the following reasons:

"1st. It gives absolute assurance of success, in exposing the material between the bottom of the caisson and the rock, and affording the opportunity of removing that material and substituting in its place concrete, which will unquestionably prevent further settling, due to the yielding material at present under the caisson.

"2d. It affords the opportunity of raising and leveling the pier in its original and proper position.

"3d. It affords the opportunity of making a thorough examination of the timbers of the crib and the caisson, and of repairing any damage done to the crib, arising from the settling.

"The report of the diver, confirmed by my own examinations, leads me to believe that some damage has been done to the upper timbers and the sheathing planks during the settlement, and I consider it important to either restore the sheathing plank or to fill the pockets to a certain distance inward on the several faces to hold the filling in the crib. This being done, the present filling, although of small stone, sand and gravel, will no doubt be sufficient and satisfactory.

"During the examinations recently made, a number of 12 by 12-in. timbers were bored entirely through at depths varying from 10 to 30 ft. below the water surface, and in every instance the timber was found to be firm and sound.

"The design shows the new caisson as constructed entirely around the old one. It may not be necessary to construct it around more than one-half of the old one; that is, around the west corner and extending the length of the side toward the north and south corners, respectively. This, however, forces us to utilize the old caisson as one wall of the coffer-dam, and it is possible, if not probable, that this will not hold the water back, so that the space between the two caissons may be pumped out. However, this half can be sunk as a section and the other half sunk, if found necessary; no change in the design will be necessary. It will be necessary to make a water-tight connection between the ends of the new caisson and the old caisson at the north and south corners. It will also be necessary to bolt two channel irons at each end of the new caisson, so that, in the event of adding the second section around the east corner and extending the south and north corners, the two sections can be united so as to form a water-tight joint.

"The estimated cost of the caisson and coffer-dam constructed entirely around the old caisson is about $40 000, and of the half caisson and dam about $20 000 to $25 000.

"The filling above the deck of the caisson required to sink the caisson may be of any material desirable, as far as practicable, the material taken out of the working chamber during the sinking may be used for this purpose, and in fact this should be required, or at any rate no payment should be made for the filling if the contractor prefers to get it from other sources; also the material between the new caisson and the old one can be dredged during the process of sinking and used for filling. This, however, should be done with judgment, as it will evidently be unwise to remove the material at and near the west corner of the old crib until we have at least given some temporary support to the caisson to the west of the center line of the pier, or that diagonal line joining the north and south corners.

"Owing to the relatively small depth of the earthy material, through which the sinking is to be done, it is possible that no filling above the deck of the caisson will be necessary. Filling this space with water may answer every purpose. It may not be necessary to fill the working chamber itself if a water-tight joint can otherwise be formed between the bottom of the caisson and the rock. If not, a concrete or clay puddle must be used of sufficient thickness to secure water-tightness. Owing to the shaly and inclined nature of the rock, it may be necessary to cut a longitudinal trench in working chamber and to fill this with puddle or concrete in order to cut off seeping water along the seams of the rock. These are matters to be determined by observation and the exercise of good judgment at the time. It is proposed to obtain water-tightness of the structure above the deck of the caisson by means of caulking the timbers and plank walls, and not by means of the filling material. One or both walls should be caulked for this purpose.

"The air-tightness of the working chamber must be secured by thorough and perfect caulking. These and other similar matters will be embodied in the outline of the specifications accompanying the drawing. You will note the upper half of the structure is so designed and connected that it can readily be removed. The lower half being below the bed of the river, or at least of sufficient depth below the water surface not to interfere with navigation, can be left in place. I, therefore, respectfully submit the accompanying drawings, marked Sheets 1, 2, and 3, accompanied with the following suggestions and recommendations.

"1st. Sinking the caisson in two sections, the one section extending from the south corner around through the west to the north corner, which may be all that will be required, and sinking the second section from the south corner around through the east to the north corner, if found necessary, owing to the open condition of the old crib.

"2d. Excavating the material from the north and south corners of the old crib and between the old and new structures to a point, say,

half way between the south and north, respectively, and the west corner, removing the loose material from beneath the old structure and replacing with concrete, or some temporary support.

"3d. Then excavating the material from, at and near the west corner, exposing the space below the old structure, removing the loose material and replacing the concrete or some temporary support. This temporary support contemplates raising the pier to its original and vertical position.

"4th. If practicable, I would suggest supporting the superstructure on temporary supports during the time occupied in cleaning out the material between the old and new structures and until the permanent filling is in place and has had a reasonable time for hardening.

"5th. For the purpose of lifting the pier, I would recommend the employment of hydraulic jacks. It is estimated that hydraulic jacks having a main cylinder of 3.5 in. diameter and a pump plunger of ¾ in. diameter, will raise 20 tons of 2 240 lb., without excessive steam pressure. Upon this basis there will be required from 30 to 35 jacks, assuming that the superstructure is temporarily lifted from the pier; the cost of this portion of the work will be little more than the cost of the jacks.

"6th. With the structure supported on jacks the removal of the loose material and substitution of concrete (for which the best Portland cement should be used) can be proceeded with without risk or danger; otherwise this work should be carried on in sections, and precautions taken not to reduce the supporting resistance, as it now exists, at and near the west corner, until the caisson is well supported, two-thirds to three-fourths of its area. The amount of concrete estimated for filling under the caisson will not exceed from 150 to 200 cu. yd.

"7th. As the design submitted will afford ample opportunity for a full examination of the caisson and crib, I would suggest filling the open spaces on the sides with concrete to the depth of from 12 to 18 in., forming a wall to hold in place the present loose and imperfect filling. A good sheeting of timber, well bolted to the timbers of the crib, will probably answer every purpose.

"The character and extent of this work necessary, can only be determined at the time.

"With the drawings you will find a bill of material, estimate of concrete required, and a detail report of the character and extent of the examinations and the information obtained.

"Trusting that you will find everything in satisfactory shape, I am,

"Yours truly,

"W. M. PATTON.

"P. O. Box 209
"BLACKSBURG, VA.

"In the event that it is deemed better or found necessary to entirely enclose the old structure with caisson and coffer-dam, it may be found more economical and satisfactory to excavate the rock at and near the south and east corners, and to lower this portion of the structure, thereby bringing the pier to its original vertical position, rather than lifting the westerly portion by means of hydraulic jacks."

REPORT IN DETAIL OF THE EXAMINATIONS MADE AT PIVOT PIER, LITTLE ROCK JUNCTION BRIDGE, MO. PAC. RAILWAY.

"*South Corner.*—Depth of water, 28 ft. 11 in. below low-water surface. Depth of rock, 35 ft. 6 in. below low-water surface. Drilled into rock 4½ in.

"The pipe was driven through fine sand, followed by a layer of coarse white sand mixed with black scales, feeling and looking like graphite; immediately overlying the rock was found a layer of fine and coarse gravel mixed with coarse sand. The rock is commonly called slate, is a black shale, which yields readily to the drill. Time occupied in setting and sinking pipe about 3½ hours.

"*East Corner.*—Depth of water, 13 ft. 6 in. below low-water surface. Depth of rock, 37 ft. 6 in. below low-water surface. Drilled into rock 3 in.

"The pipe was easily driven in medium sand, passing into sand mixed with gravel to a depth of 23 ft. below low water. This continued to a depth of 30 ft., where sand and gravel mixed with a soft but tenacious clay was found. At 34 ft. down a rather fine sand was found, mixed with clay. Rock was reached at 37 ft. 6 in. below low-water surface. There was found no well-defined stratum of gravel at this corner. The rock was of the general character, as already indicated. Time occupied in sinking about 4 hours.

"*North Corner.*—Depth of water, 22 ft. 7 in. below low-water surface. Depth of rock, 39 ft. below low-water surface. Drilled into rock 3½ in.

"Pipe was easily driven through medium sand to a depth of 29 ft. 3 in. below low water. At 26 ft. 9 in. was found fine and coarse sand mixed with fine and coarse gravel, and black scales, which continued for about 2 ft. to a depth of 28 ft. 9 in., the driving being somewhat difficult to a depth of 30 ft. 9 in., below this the pipe entered a fine flowing sand, which rapidly filled up the pipe for several feet whenever the jet was stopped. This material consisted of fine white sand, suggesting quicksand by its readiness to flow. From this point it was necessary to run hammer and jet simultaneously. The large pipe followed the small pipe readily, but would fill up if jet stopped. This fine sand continued to a depth of 39 ft. Drilled into the rock about 3 ft. Rock similar to that at other places. Time about as at other points.

"*West Corner.*—At and near this corner large quantities of rip-rap have been deposited. It was impracticable to find an opening through this sufficiently large to permit the passage of the 3-in. pipe. The small jet pipe was easily run to a depth of 33 ft. 6 in. below low-water surface. Further progress was prevented by a bed of gravel and the friction on the pipe above. Several attempts were made to force the 3-in. pipe through the rip-rap, which invariably resulted in the squeezing and mashing of the large pipe and the parting of the small pipe. On Saturday, March 12th, however, we succeeded in getting the 3-in. pipe through the rip-rap.

"Depth of water 19 ft. 3 in. below low-water surface.

"At 24 ft. down, the pipe brought up on a log. This log was bored through, it proving to be 12 in. thick. The timber was found to be firm and sound. The 3-in. pipe was driven through this log, and at 26 ft. 10 in. the pipe struck another log, which was partly bored through, when further work was stopped at 6 P. M. It had been raining hard all day, which continued through the night. The river rose rapidly, so much so that it was necessary to remove the barge to a place of safety. Work in the river was then abandoned for one week. This ended the first week's work. There were many delays, caused by time required to cut and thread pipes, and pump getting out of order, foaming of the boiler, etc. These were, no doubt, incident to and unavoidable in this kind of work, and it is not intended to attach any blame to any one, as all parties seemed disposed to do anything when called upon, and on the whole I was entirely satisfied with the work done the first week.

"Seeing no prospect of getting to work in the river, preparations were made to get into the crib within the well in the center of the pier, and on Thursday, March 17th, work was commenced. The well was found to be filled to about 11 ft. with silt, logs, and stone. After several efforts, a 3-in. pipe was forced to the timbers of the deck, and these timbers, about 2¼ in. thick, were bored through, admitting the small pipe without difficulty several feet below the deck and into the filling of the crib. This filling proved to be small stone, sand, and gravel. In attempting to drill through a stone the small pipe parted, resulting in loss of drill.

"On Monday, March 21st, the river having fallen a few feet, an effort was made to resume work in the river. A large portion of the day was consumed in getting the barge in position, and little was done subsequently, owing to the almost continuous foaming of the boiler, and consequent failure to supply requisite pressure. On Tuesday, with a rising river, a new pipe was sunk near the west corner, but it could not stand the constant vibrations caused by a strong current acting on some 40 ft. of unsupported length, and broke. At this

time the river had risen considerably, and it was deemed necessary to remove the barge to a place of safety.

"I desire to express my appreciation of the uniform kindness, consideration and faithfulness of officers and laborers on this work.

"With great respect,

"W. M. PATTON."

SPECIFICATIONS FOR THE CONSTRUCTION AND SINKING OF CAISSONS FOR THE PURPOSE OF REPAIRING THE PIVOT PIER AND PIER 4 OF THE LITTLE ROCK JUNCTION BRIDGE, MISSOURI PACIFIC RAILWAY.

"There will be constructed two caissons, the one for the pivot pier and the other for Pier 4. These caissons may be of such design and dimensions as will entirely surround the old piers of the bridge or will extend around any portion of these piers, as may be directed by the Engineer of B. & B. of the Mo. Pac. Ry. These caissons shall be constructed according to designs given on Sheets 1, 2, and 3 accompanying and forming a component part of these specifications, or according to such other design as may be accepted or approved by said Engineer of the Railway Company.

"Sheets 1, 2, and 3 give plan, elevation, section, and exterior and interior detail of the caisson and coffer-dam to be constructed entirely around the pivot pier.

"The structure is built of timber and iron; all timbers are 12 by 12 in. in cross-sections, excepting a few pieces 9 by 12 in. and 6 by 12 in., used for supporting braces, and 3-in. planks necessary for lining the caisson and coffer-dam. Exact lengths are given and where not given variable lengths are allowed. All necessary dimensions are given on the drawing and require no description. In the event that Engineer determines to construct a caisson of only one-half the dimensions given in the plan, no change in design will be required except closing up the free ends as indicated in the drawing.

"*Timber.*—All timbers used in the construction of the caisson and coffer-dam may be what is known as merchantable, and may be either pine or oak. *The outside verticals forming the lower half of the structure must be of the best long-leaf yellow pine and showing clear heart on the exposed face.* All timbers must be free from all defects that will impair their strength, such as rot, dotiness, or sponginess, large knots, or deep shake, or cracks of any kind. All framing and fitting must be done in a workmanlike manner and to the approval of the said Engineer.

"The upper and lower halves of the structures are to be built entirely independently and not bolted together in any manner. The two halves being held together solely by the 1¼-in. hook-rods, as indicated in the drawings. The lining plank in the caissons, as well as

in the coffer-dam, shall be planed on the edges for a caulking joint.

Caulking.—The caulking of the inner plank of the air or working chamber shall be thoroughly done, so as to make the sides and top of the air chamber completely air-tight and paved over with tar or pitch. Oakum shall alone be used in caulking. The joints of the top deck course shall also be sufficiently caulked to form a water-tight surface. The verticals forming the sides of the caisson, or lower half of the structure, shall be caulked on the outside, so as to form a water-tight surface. The inner plank lining of the coffer-dam shall be caulked, so as to make water-tight surfaces, as reliance for keeping out water is placed entirely on this caulking.

"*General Construction.*—The exposed surfaces of the lower half of the structure are built of timbers 12 by 12 in. by 23 ft., thoroughly bolted to these on the inside are five layers of horizontal timber, one layer on top of the other, which are well bolted to each other; spiked to these horizontal timbers are two layers of 3-in. plank, the first course placed diagonally and the second or inner vertical and caulked. Resting on top of the horizontals and plank, is placed a solid layer of 12 by 12-in. timbers, bolted at the ends to the horizontals; on top of this another solid layer of 12 by 12-in. timbers, placed diagonally and bolted at intervals of about 5 ft. to the first course, with 1-in. square by 22-in. drift-bolts. Over this course another solid course of 12 by 12-in. timbers laid longitudinally and similarly bolted to the course below. These three courses constitute the deck of the caisson proper. To the underside of this deck a layer of 3-in. plank is spiked and caulked. Above the deck and between the vertical sides, an open crib of 5 courses of timber is constructed, bolted to the verticals, to the deck and to each other. Above the crib is one set of cross-braces between the verticals and resting on longitudinals, bolted to the verticals. Diagonal rods are used in spaces between these braces and in a horizontal plane. Another set of cross-braces are placed at the top of the outside verticals. This completes the lower half of the structure and may be denominated as the caisson.

"*Coffer-Dam.*—Sills are placed on top of the verticals on both walls of the caisson, and upon the sills, at a general interval of 5 ft., center to center, vertical posts 12 by 12 in. by 21 ft. are placed, and on top of these posts caps are placed. These posts may be spiked or bolted to caps and sills, or they may be mortised and tenoned. Two layers of 3-in. plank are then spiked to caps, sills, and posts on the inside. The first layer is placed diagonally and the second or inner layer horizontally and caulked.

"Cross-pieces are then placed over the caps and projecting 18 in. beyond outside verticals, through which the 1¼-in. rods with hooks, pass. These rods are connected with the I-bolts, let into the verticals. Plate washers are used at the top, as the two halves, upper and lower,

are to be pulled and held together by these hook-rods. The two joints between sills and top of the verticals of the caisson are to be well caulked after the two portions are pulled firmly together. An inner strap, countersunk, encircles the caisson, to which it is well spiked just above and resting against the I-bolts. There are three sets of braces between the walls of the coffer-dam, with diagonal rods between them and horizontal planes, as indicated in the plan and sections. These rods have swivels or couples at some intermediate position.

"The coffer-dam is framed in four sections, two of which are 70 ft. long and two are 44 ft. long. These sections are connected at the ends by diagonal rods with swivels and couplings, double posts being used at the junction of the sections; also 18-in. channels are let into, but not bolted to, these posts, in order to hold the sections together.

"Removing these channel irons and uncoupling the diagonal rods, sever all connections between the sections of the coffer-dam as well as the two walls of the same section, so that the upper half of the structure can be readily removed at the completion of the work. The lower half of the structure is intended to remain permanently in place. All other details, with bolts, straps, and braces, are shown on the drawings, and also the dimensions of the same, as well as the bill of material accompanying.

"*Iron.*—All iron is to be good, tough, ductile, and fibrous, except that all washers may be cast iron. The top washers for 1¼-in. hook-rods should be wrought-iron plate, about 1 by 6 by 6 in. In all cases the grip of screw bolts, that is, net length of washers, is billed. Proper allowance must be made for heads, nuts, and washers. The weights given are for full lengths, including heads, nuts, and washers. All bolts must be wrapped with oakum under head before driving, and at the threaded ends before placing nut and washer on.

"All drift-bolts are billed as 1 in. square. Round drifts 1 in. in. diameter may be substituted, except for those bolts used in the outside verticals of the caisson.

"*Filling Material in Caisson and Coffer-Dams.*—All these caissons are only used for a temporary purpose and for convenience and certainty, and, after being sunk to the proper depth, serve only the purpose of coffer-dam, the material used in the filling is of secondary importance. The filling in the coffer-dam is only required to give weight necessary to sink the caisson and may therefore be any material. As far as practicable, the material taken out of the working chamber may be deposited above the deck of the caisson and in the coffer-dam, thus furnishing weight to sink the caisson. Also the material between the old caisson and new caisson may be excavated by dredging and likewise deposited in the coffer-dam, care being taken, however, not to remove the material at and near the west corner of the old caisson until it is safely secured by permanent supports, or

temporary supports, from further settling. It may not be necessary to entirely fill the coffer-dam in order to secure sufficient weight. If necessary or desirable, water may be let into the coffer-dam so as to supplement the weight of other material, in such case, the water can be ultimately pumped out, thus facilitating the removal of the coffer-dam.

"The filling in the working chamber of the caisson after it has reached the rock, or its final position, is only necessary to prevent leaking under the caisson. It must be such material, and in such quantity, and so placed as to prevent any under flow of water. No filling of any kind may be necessary, and will not be if the water surrounding the old caisson and between it and the new caisson can be pumped out and kept out. In this case some additional cross-bracing in the working chamber may be found necessary. It may be found necessary to fill the working chamber in the whole or in part with clay puddle, a water-tight joint between the caisson and rock being the important object.

"*Excavating the Material Between the Old and New Caissons.*—This may or shall be done in part during the sinking of the caisson, and the material thus excavated used to weight the caisson. In any event, the material at and near the west corner of the old caisson must not be disturbed or removed until a sufficient and safe support has been placed at other portions of the caisson. The material at and near the west corner can then be removed, thus exposing the entire surface of the old crib and caisson from its top to rock.

"*Raising or Leveling the Old Caisson.*—It is the intention of the Railway Company to lift and return the pier to its original and proper position, for which purpose the use of hydraulic jacks will be necessary. It will be necessary for this purpose, or at any rate may be advisable, to support the superstructure on trestles or some other form of temporary supports. The posts of the coffer-dam should not be used for this purpose. However, a sufficient number of additional posts may be inserted in the coffer-dam to carry the load. It will be better and safer, however, to support the superstructure by supports entirely independent of the coffer-dam.

"After inserting the jacks, with proper and safe bearing for uniform distribution of pressure, it may be found necessary to flood the coffer-dam, thereby reducing the weight to be lifted. In this case the pumps will have to be above the water surface and connected by means of small pipes to the main cylinders of the jacks. An additional number of jacks should be provided, and as the water is again pumped out, these should be brought into bearing so as to support the extra weight brought into action when the water is removed.

"In the event of constructing a caisson entirely around the old structure, it may be easier to excavate the rock from the south corner around through the east corner and as far as may be necessary toward

the north corner, and to level the structure by lowering the high portion of the caisson, rather than by raising the lower portion by means of hydraulic jacks.

"*Filling Under Old Caisson.*—In whatever manner the old caisson is restored to its proper position, or whether it is left in its present position, all loose material is to be removed from beneath the old caisson and good Portland cement concrete substituted. The concrete shall be composed of such material, mixed in such manner and placed and rammed in accordance with the approval and direction of the said Engineer of the Railway Company. Unless the structure is safely and securely supported by jacks, this work of underpinning must be done in sections, so as at no time to endanger the structure by further settling at any point. The jacks, if used, must be left in position until in the judgment of the Engineer the concrete has been sufficiently hardened to be able to carry the load with safety. The jacks can then be removed and the spaces filled with concrete, if the Engineer may so require.

"*Refilling Between Old and New Caisson.*—The space between the old and new caisson, from the rock to the bed of the river, will be refilled with material, if so required by the Engineer.

"*Removing the Coffer-Dam.*—The upper half or section of the structure, known as the coffer-dam, will be removed as already indicated.

"*General Remarks.*—In the bill of iron no mention is made of shafts, pipes, or other appliances required to sink the caisson, as these are regarded as belonging to the contractor's plant.

"The contractor shall furnish all material, tools, machinery, and apparatus of all kinds required to construct and sink the caisson, pump out water, and remove material. The contractor shall be responsible for any damage to the old structure, and shall take all risks incident to such work. The Railway Company shall be in no wise responsible for loss of life, damage to property, or interference with the safe navigation of the river. He will be responsible for all acts of his employees, and shall discharge any of his employees, when so directed by the Engineer, who fail or refuse to perform the work in accordance with the direction of the Engineer or his duly authorized representative.

"All material is to be approved and accepted by the Engineer before being used in the work. All work must be done in a thorough and workmanlike manner and to the approval of the Engineer or his duly authorized representative.

"*The Caisson for the Pivot Pier.*—The caisson for the pivot pier is composed of four sections; two of these are 70 by 13 ft. in plan and the other two are 44 by 13 ft., when the caisson entirely surrounds the old caisson. If it is determined to enclose only one-half of the old caisson, only three sections will be required: one 70 by 13 ft.;

and two 38 by 13 ft. All sections are 46 ft. 3 in. from bottom of caisson to top of coffer-dam, and 47 ft. 3 in. over all. Estimated cost on first plan $40 000, and on second $21 200.

"*The Caisson for Pier 4.*—The caisson for Pier 4 is composed of four sections; two of these are 70 by 13 ft. in plan and two are 27 by 13 ft., when the caisson entirely surrounds the old caisson. If it is determined to enclose only one-half of the old caisson, only three sections will be required, one 27 by 13 ft. and two 35 by 13 ft.; all sections are 46 ft. 3 in. from bottom of caissons to top of coffer-dam and 47 ft. 3 in. over all. Estimated cost on first plan $34 000, and on second $17 000.

"In either case the designs are essentially and substantially the same, only differing in axial length."

APPENDIX B

LETTERS OF A. J. TULLOCK ACCOMPANYING BID OF MISSOURI VALLEY BRIDGE AND IRON COMPANY FOR CARRYING OUT PATTON'S RECOMMENDATIONS AT PIVOT PIER AND PIER 4.

"LEAVENWORTH, KAN., Aug. 17, 1898.

"MR. E. FISHER,

"Engineer B. & B., Mo. Pac. Ry.,

"Pacific, Mo.

"DEAR SIR: Referring to your claims and specifications for the repairs of Piers 2 and 4 of the Little Rock Junction Railway Bridge, will say that I have gone into that subject quite fully and submit you herewith three separate propositions, covering the main portion of the work to be done in said repairs, leaving, however, certain parts of the work, the nature of which cannot now be determined, to be arranged at a later date when it can be clearly seen what is required. Such work can very well be done on a percentage basis, and probably so to best advantage.

"It is quite clear that the settling of the piers mentioned in the Little Rock Bridge arises either from the caissons not having been originally properly founded in solid rock, or from the crushing of the timbers in the cribs between the caissons and the masonry, as it is well known that these cribs were not well constructed when the work was built. Present appearances would indicate settlement due to the caissons not having been properly founded in the solid rock, rather than to the crushing of the cribs, but as against that, we have the written statements of the contractor who built the work, Mr. Barr, and of the Resident Engineer in charge of the work, Mr. Purdon, both reputable gentlemen, both of whom agree in saying that the caissons were properly founded in bed-rock and that the settlement of these two masonry piers must be due to crushing of the cribs. However, this can only be satisfactorily determined by actual examination, after sinking outside surrounding caissons as is now proposed.

"From a careful reading of your specifications, I conclude that, inasmuch as no concrete or filling of any kind is required in the caissons, that it was the intention of Prof. Patton that these caissons should be used simply for the purpose of getting down into the rock around the old piers to permit the underpinning or other repairs suggested by him, and that he contemplated relying entirely upon the underpinning or repairs made directly in the body of the old piers for permanent stability.

"It seems to me that we cannot safely rely on such underpinning for safety in this case, and it is very doubtful indeed if such underpinning can be done at all, without greatly endangering the entire

structure of the old piers. It is my opinion that the work to be done in this case should properly consist of sinking the caissons as proposed in your specifications, but that they should be filled with some material sufficient to provide permanent lateral stability, and that the space between the new caissons and the old piers should be excavated down to bed-rock, pumped out, and filled with concrete to a sufficient height to thoroughly support the old pier and prevent any lateral movement. This being done, and the outer caisson being sunk into rock so as to absolutely shut off any outward movement of soft material, which may underlie your present pier, the absolute stability of the pier against further settlement or movement of any kind will undoubtedly be assured, providing this settlement is due to lack of perfect foundation under the original caissons. I have therefore arranged my proposals with a view of carrying out this plan, as well as making them applicable to the programme apparently contemplated in your specifications, in case when the excavations are made, it might be found possible to make the repairs as suggested by Prof. Patton.

"I will not hesitate to say in advance, however, that I consider the method of repairing proposed entirely impracticable, and too dangerous to warrant its adoption, except under conditions so extremely favorable that we have no right to assume they will exist in this case.

"The caissons necessary to surround these old piers properly are necessarily so unusually large as to become very expensive as compared with the ordinary caissons used underneath piers of that kind in original construction, and in addition to the fact that a very large volume of material, both in construction and excavation, is involved, the peculiar form of these caissons is such that they must be very strongly constructed and braced in all directions to prevent danger of their becoming twisted in sinking, and being wrecked in that way. The cost of these caissons in place, therefore, becomes very much larger in proportion than the cost of ordinary caissons for new piers.

"We have figured out these caissons in detail, and submit you herewith blue prints of same, numbered respectively 3 928 and 3 929.

"I submit the following proposals for doing this work:

"*Case 1.*—This case includes furnishing, building, and sinking to the depth specified by you, the timber caissons and coffer-dams described in your specifications, and shown on our drawings, including the excavation of all material within the caissons to such specified depth, and the excavation and pumping out of the material between the new caissons and the old piers down to bed-rock, providing it can be excavated that far without endangering the old piers. This case, however, does not include the filling of any of the caissons with concrete.

"Furnishing, building, and sinking as above described,

"Pier 2 (draw pier)..... $28 600.

Furnishing, building, and sinking Pier 4... 25 600.

"*Case 2.*—Caissons and coffer-dam constructed and sunk precisely the same as in Case 1, and all necessary excavation same as in said case, adding thereto, however, the sealing of the caissons in the working chambers with Portland cement concrete to a height of 2 ft. above the cutting edge, all the way around, and remainder of the working chambers and caissons to be filled with pure sand:

"Pier 2, as above described.....$31 600.
Pier 4, as above described..... 27 950.

"Steel caissons 6 ft. wide in the clear, height same as for wooden caissons, Cases 1 and 2, surmounted by wooden coffer-dams as in Cases 1 and 2—these caissons to be sunk to the same depth as the others and to have the entire working chambers of same filled with Louisville cement concrete, the remainder of the caissons to be filled with sand. Excavation between caissons and piers same as in Cases 1 and 2. This case presents some advantages and would perhaps be preferable to the wooden caissons, but for the fact that it may be practically impossible at the present time to get the plates necessary to construct these caissons of steel quickly enough to insure the work being done within the low-water period in the river. I have only considered this case within the past 24 hours, and am therefore unable to submit you a complete drawing of the steel caissons, but hand you herewith a pencil sketch which will serve the purpose of showing the construction contemplated. This caisson being only 6 ft. in width at the bottom, while the wooden caisson is 12 by 13 ft., it will serve the same purpose so far as shutting off the outflow of material from underneath the old pier is concerned, but not having so wide a base, it has been considered best to fill the entire working chamber of these caissons with concrete, whereas in the other case, with the wider base, we can probably, with perfect safety, use part concrete and part sand as described in Case 2.

"Pier 2, as above described.....$30 770.
Pier 4, as above described..... 26 655.

"In connection with this work, I will furnish and put in place the concrete filling required for the spaces between the new caissons and the old piers, at the following prices per yard:

"For Louisville cement concrete.....$4.85
For Portland cement concrete..... 7.38

"In all of the concrete work herein contemplated, the proportions for mixing are assumed at 1, 2, and 4 for Louisville cement concrete, and 1, 3, and 6 for imported Portland cement concrete. The quality of Portland cement assumed to be equal to Alsen's German.

"In this connection, I wish to say that we have a plant on hand ready to do this work, and could commence it immediately, our plant being now idle at Jefferson City, Mo. I will be glad to do any extra

work that may be required, not covered by these proposals, on a basis of cost plus 10%, including, of course, the equipment costs as well as labor costs.

"For the purpose of comparison, and illustrating the expensive character of the repair work contemplated on these two piers of your bridge, will say that I should be glad to furnish all material and build an entire new set of piers for that bridge according to the best modern specifications and practice, for the sum of $125 000.

"Very respectfully submitted,

"A. J. TULLOCK."

"LEAVENWORTH, KANS., Aug. 18, 1898.

"MR. E. FISHER,
"Engineer, B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"DEAR SIR: Referring to the caissons for the repair work of your piers at Little Rock, we figure the cost for sinking the extra depth on these caissons, assuming that they are to go through bed-rock, at about $800 per foot for the large caissons for Pier 2 and about $750 per foot for the smaller caissons Pier 4. We assume, of course, that if this extra depth is required, that the excavation will be practically all solid rock, which makes the cost run up quite high. I have to-day carefully examined the record of the borings made by ourselves a few years ago, and also those made as reported in your specifications, and I feel quite certain that the depth you have specified will prove to be about right, and that no extra depth will be required. Certainly none will be required, unless we should strike a fissure in the rock, which it would be necessary to shut off, and even then in that event, we could probably clean it out and plug it with concrete inside the caisson much cheaper than sinking the whole caisson farther down. In figuring the cost of the entire work, we have distributed the plant charge, machinery, repairs, maintenance, use of plant, and all charges of that kind over the depth specified, and as this charge would remain the same, or practically the same, for a less depth, the saving for sinking less than the depth specified would not be a great deal, particularly if we assume that down to that depth only a portion of the excavation is rock. We figure, therefore, that the saving, in case we do not go as deep as specified, would be about half the price named per foot for extra depth.

"The price which we name for extra depth would only apply for a depth of 5 ft. below the depth specified in your specification. For any further depth, there will be an increase of about 50 per cent. This increase for extra depth is partly due to the fact that the schedule of wages paid the pressure men or "sand hog" increases for every foot below 20 ft.

"In reference to completion of the work, we would expect to complete the entire work in four months, providing we receive the order for it immediately, and I will undertake to do it in that time, barring accidents or floods or any such things as would be beyond the control of the Contractor. I feel certain, however, that nothing is likely to occur that would stand in the way of the entire completion of the work in about four months, and we have made our figures on that basis. If it took much longer than that time, we would be the principal losers, as the cost would increase very fast by the extra time used.

"In going over the figures on this work to-day carefully, I find that we have made them exceedingly close, and if we do that work, we should want to know at once about it, or within the next few days at farthest. I shall be glad, therefore, if you will reach a conclusion as soon as possible and notify us.

"Yours truly,

"A. J. TULLOCK.
"Proprietor."

"LEAVENWORTH, KANS., Aug. 19, 1898.

"MR. E. FISHER,
"Engineer, B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"DEAR SIR: I have your favor of the 17th inst. with reference to modifying the dimensions on caissons for repairing the piers of your Little Rock Bridge, and have carefully noted what you say. I hardly think that it would do to figure on reducing the size of these caissons, nor do I think it would be desirable to do so. In the first place, we have no greater width between the caisson and the pier than is necessary to properly support the pier. In the second place, we cannot very well work closer to the old pier safely than 6 ft., as by getting much closer we would be likely to encounter more serious difficulties than are now contemplated and add to the cost. Further, in caissons of this character, on account of being so unwieldy and difficult to handle, it is not likely that they can be kept in exact position; in other words, the caisson is liable to be from 6 to 12 in. out of position in almost any direction, and this perhaps cannot be avoided. Of course, we will endeavor to keep it in position, or if out of position at all, to have the greatest distance between caisson and pier on the side to which the pier leans, so as to have the heaviest supporting wall on that side. We might save some caisson material by cutting down the dimensions of the caissons a little, but we would at the same time, increase the cost per unit quantity in certain parts of the work, so that on the whole there would not be much saving. It will be a difficult job at best to keep the wooden caissons of that character from racking to pieces on account of uneven support, even with a width of 12 or 13 ft., and

the more we reduce its width the greater this difficulty becomes. So far as the caisson itself is concerned, and as to the opening between the caisson and the old pier, if we get much closer to the pier than we have figured, we will increase the cost of sinking more than the saving made in caisson material by reducing its exterior dimensions.

"In the case of the Kansas City draw-pier, where this plan was followed with a circular caisson constructed of steel, the caisson was brought closer to the pier than in your case, being only 3 or 4 ft. from it, but in sinking that caisson, the average progress per day was from 3 to 5 in. for each day of 24 hours, and the entire cost for that pier was between $40 000 and $50 000, being somewhere close to the latter sum. We have studied the subject quite thoroughly and carefully from an engineering standpoint, and also from a practical standpoint with Foreman Stewart, who has had large experience in such work, and it seems to me that we have hit upon the best construction for the place at the least possible cost. I realize fully that this is a good deal of money to put on repairing two old piers, but I do not see any way to better the situation unless you should decide to put in entire new piers throughout the whole bridge, and in answer to your inquiries about that project, will say that the price I gave you of $125 000 for replacing the entire piers contemplated putting the further pier on the Argenta side on piles, which could probably be safely done. However, we could make that pier a pneumatic pier also putting it down to rock, by increasing the cost to $135 000, or thereabouts, and of course in these new piers we would not expect to use Cabin Creek stone except for backing, the face stone to be strictly first-class masonry, both in quality of stone and workmanship, instead of cheap rubble, as in the case with the present old piers of that bridge.

"I find that I shall probably have to be in Chicago on Monday on some bridge business for the Santa Fe, but can be in St. Louis Tuesday, or if you wish to take up the subject Monday and could wire me to-morrow, I might postpone my Chicago business until Tuesday. Should you decide to have us do this work, I would thank you to have Mr. Spoor, your Timber Agent, give me the names and addresses of the mills which could furnish this timber quickest and best, as it would be necessary to move at once in the matter of getting timber and equipment on the ground in order to be sure of getting the work done before there is danger of high water.

"Very truly yours,

"A. J. TULLOCK.
"Proprietor."

APPENDIX C

LETTERS OF A. J. TULLOCK, WHILE CARRYING OUT WORK RECOMMENDED BY PATTON.

"ST. LOUIS, November 24, 1898.

"E. FISHER, ESQ.,
"Engineer, B. & B., Missouri Pacific Railway,
"Pacific, Missouri.

"DEAR SIR: We have the caisson of the center pier built up about 8 or 10 ft., and last night we placed the second course of decking on the caisson. It is already lowered, so that one-third of its weight is taken up by water displacement, thus relieving the lower screws and supporting piles. We are, therefore, safely past one state of danger, which arose from the fact that our piles did not have much penetration and being difficult to brace. The matter of supporting so large a caisson on lowering screws was naturally a deal of a problem and somewhat risky. We feel, however, that we are now safely past any danger from the trestle support, and we shall, within the next two days, unless something unexpected interferes, have the caisson resting entirely on water and the screws relieved and removed to Pier 4. As soon as the screws can be set up on Pier 4, we will proceed with the building of the caisson, while we continue the sinking of the caisson at Pier 2. As near as I can tell, we should have air in the caisson of Pier 2 in about ten days or two weeks at the farthest. This, however, depending somewhat upon how much trouble we have in getting a proper bearing for this large caisson on the bed of the river. You will readily understand from the shape of this caisson in its unusual dimensions, that it becomes very necessary to handle it gently and lay it on the bottom in a good and reasonable uniform support, in order to prevent it from being strained and the joints opened so that it will not serve its purpose. Very little irregularity of support when the caisson commences to be loaded would likely spring it badly, notwithstanding the fact that we have built it unusually strong. It, therefore, becomes very important to get it carefully landed and this is our next critical movement. The fact that the sand has all been scoured away from that pier, which we did not expect, increases this difficulty very greatly, and it is impossible to make a fill around the pier by wing-dams above, as there is not enough material carried by the water at present to make such a deposit practicable. We have to depend, therefore, upon leveling off as much as we can before resting the caisson on the bottom, and then using sacks of sand for building up underneath the cutting edge of the caisson for support at the low points, and with this in view, I have already shipped several thousand coffee sacks from St. Louis to Little Rock so as to be in readiness. Superintendent

Stewart has the work well in hand and understands thoroughly every move to be made and every emergency likely to be met with.

"We are now approaching a time on this work when we must think carefully about what you are going to do when these caissons are down in position and the excavations made between the new caissons and the old piers. Of course, you cannot tell exactly until these excavations are made, but you must contemplate to some extent in advance, what material will be required, so that this material, such as Louisville cement and crushed rock, can be procured, on hand ready for use. You will recollect that while our contract provides a price for the concrete filling of this area between the new caisson and the old pier, the work of so doing is left for later determination. Therefore, I have provided no materials for such filling, and can only do so on your order. I would suggest that you authorize me to procure a certain amount of Louisville cement and crushed rock, which you think will safely be needed, immediately, so as to have it on hand when we are ready for it. You can certainly approximate the minimum quantity needed, and it is more than likely that you will want to fill this whole area as far as possible with concrete, in order to get the best results. Now, while it is important for the Contractor, on account of the coffer-dam and caisson and danger of floods in the river, that this work should be dispatched very rapidly after the excavation is made, it is still more important for the Railway Company and the owners of the old bridge that the structure should not be left exposed a *single hour longer than is absolutely necessary after this excavation is made.* There are abundant reasons for this, which you will doubtless recognize at once, and I now only mention one or two of them. In the first place, that old pier, caisson, and crib was so poorly constructed at the beginning that even if it were in as good condition as when first built, it would be very bad judgment to allow it to remain any length of time with the material all excavated away from it, as we are required to excavate. There is always danger in such performance of something of the unexpected happening, even if the pier stands perfectly square and true and is well built. Again, the timbers in that old caisson and crib, which are rough hewn timbers, improperly framed and about half enough in quantity, have now been soaking in the river about 14 or 15 years, and there is no telling what effect exposure to the air, for even two or three days, might have on these old timbers, heavily loaded as they are, without being reinforced by any interior filling of concrete or any other material of value. It has probably come to your notice that certain kinds of timber long submerged in water when taken to the surface, apparently in excellent condition, come to pieces very quickly after exposure to the air. I do not know that there is any danger to the timbers of these old piers, but at the same time it is possible that there may be and it is a matter

worth thinking about. So on the whole, it is undoubtedly the best policy to be ready to act quickly when the excavation is made, and to get the filling with concrete made around the old pier as high as possible and within the shortest possible time after the excavation is made.

"In the case of Pier 4, I have a suspicion that that pier is already taking a good deal of its lateral support from the material, sand, etc., which now surrounds the old crib and caisson. It is possible that it may be held directly in position by this material, which you will notice extends up quite high in that pier. This being the case, we will have to proceed very carefully indeed with our excavation, both in sinking the caisson and in making the excavation between the new caisson and the old pier, as that portion will necessarily be attended with a great deal of danger, and conditions may be discovered as the work progresses, particularly with reference to the excavation between the caisson and the pier, which would make it necessary to change your plan of operations materially. I mention this now only that you may be thinking about it in advance, as I have been for some time past.

"In this connection, you will see at once the necessity of having very frequent observations taken as to the movements of these piers from the time we commence excavating, even in the new caissons alone, until we are through with the work. It is altogether likely that at certain stages of the work, it will be necessary to take these observations several times a day, and even almost hourly, so that any movement whatever of the old piers will be detected and made known to our men in charge. I mention this now so you will make arrangements with your Assistant Engineer, who has been taking these observations, to be present on the work when our excavations commence, ready to report promptly any appreciable movement of the piers. I do not anticipate any movement of consequence at the center pier (Pier 2) but of course that depends on what we may find the conditions to be underneath. I have a young office engineer, Mr. St. John, on the ground, who has experience in the use of field instruments, and I shall be glad to have him assist your engineer in taking these observations when the time arrives. Please let me know in good time if you will arrange to have your engineer present to look after this matter during our excavations. It is important that this matter should receive careful and accurate attention. Write me at Leavenworth.

"Very truly yours,

"A. J. TULLOCK."

"MR. E. FISHER, "MAY 16, 1899.
"Engineer, B. & B., Missouri Pacific Railway,
"Pacific, Mo.

"DEAR SIR: Your favor of May 6th, written at Little Rock, Ark., was duly received during my late absence. I note your instructions in

reference to the work remaining to be done at Pier 4, that is, the concrete filling, etc., I believe the plan which you propose, which is substantially that adopted at Pier 2, is a very good one, and will get the best results with the least possible expense. I think the idea of supporting the old crib as high up as possible with concrete, as we have done at Pier 2, is the best solution of this difficulty, and I also believe that the material below the concrete, if of a reasonably substantial character, such as sand or rip-rap, is equally good when covered with a sufficient amount of concrete on top, which permits you to give the desired support to the old crib, and pier with the use of a minimum amount of concrete, much less in fact, than was originally contemplated. We will, therefore, probably have considerable extra cement left on hand, which your Company can no doubt dispose of or use to advantage.

"I note what you say in regard to pumping down and excavating between the coffer-dam and the old crib, and in reply will say that I will be glad to pump down and excavate just so far as it is practicable to do, but cannot undertake to go farther. This is precisely the conditions of our contract, and you no doubt fully realize that any attempt to do more than is practicable to be done in this case, would not only be a needless waste, not contemplated, but might also, particularly in the case of Pier 4, greatly endanger that pier. You will recollect further that when you had plans and specifications prepared for this work by some eastern professor or school teacher, your specifications provided for doing a whole lot of theoretical experimental work which was absolutely absurd, and which could not possibly have been carried out under any condition. I called your attention to this at the time, and I think that you agreed with me at that time in reference to it, and I suggested the present plan of work in my proposal, which was adopted. When the contract was drawn by your attorney, which was done during my absence and afterward presented to me for signature, I called attention to the fact that in drawing it, your attorney had provided for what was probably impossible, explaining that it would probably be impracticable to do certain work which seemed to be contemplated in the contract, the character of which was somewhat the same as you had in the professor's original specifications. Your attorney then modified his language in the contract by adopting my own expression in regard to pumping and excavating, which was that we should do just so much as was practicable to be done. Now this is the meaning of our contract, and I wish to make it clear to you, as you seem in your letter to want to swing a club over the head of your contractor, by talking about compelling him to pump down below the cutting edge of the old crib and caisson and do certain other impossible and absurd things, and you know very well that there is no object whatever in doing this, even if

it could be done, and if that material, which is in this case mostly all rip-rap stone, could be taken out, you would simply have to put it back again to give the necessary support to the pier or to substitute concrete in its place, which would cost your company several thousand dollars more than the present arrangement and would be practically no better. Furthermore, you must be aware that the inclination of Pier 4 is so severe to one side that it would be extremely dangerous and inviting disaster to excavate that material between the new caisson and the old crib to any great depth. This is all clearly manifest. I am writing you thus fully as I wish to make my views clear on the subject and wish to put them on record. In sinking the caisson around Pier 4 we went down as you know on all sides through rip-rap most of the way. There was a little less rip-rap on the down-stream side of the pier than on the up-stream side, but nearly all the filling, at least, the main mass of the material around the old crib and between it and the coffer-dam, is now rip-rap, and by putting a good solid bed of concrete on top of it, you will have supported that pier in the best possible manner, as with the new caisson in position it cannot get away.

"I dislike very much to complain, but I must say that the whole tenor of your demands and instructions seems to be of the nature of threats of putting your contractor to unwarranted costs and expense rather than for the purpose of getting the best results on work or of giving any special benefit to the job or your company. You seem to lose sight entirely of the dangerous and costly nature of the work we have to perform around these two piers successfully, and of the great cost and heavy loss which has been sustained in carrying out this work safely. You seem to be looking only for a chance to put your contractor to needless and foolish expense of one or two hundred dollars in doing something which you imagine you have a right to demand, whether it is any value to the work or not, and to entirely ignore the fact that in sinking these tremendous large caissons around your old piers through rip-rap, piles and everything else, and through a large amount of solid rock, we have been compelled to proceed so slowly in order to avoid injuring your old piers, that the cost of doing the work has been double what was contemplated.

"I note that you say in your letter 'before concrete is put in, the depth of the cutting edge must be determined.' Now, when we have pumped down between the coffer-dam and the old crib of Pier 4 and have gotten that crib ready for concreting, we must be ready to concrete at once and finish quickly, as it will be dangerous and absurd to endeavor to hold that pumped out coffer-dam against chances of rises in the river and other disasters any length of time, for a foolish effort to try and determine the depth of the cutting edge of the old caisson. At best, the determining of the elevation of this cutting edge is only to gratify a curiosity and can serve no purpose.

Whatever its location might be found to be, if possible to find it, it would not in any way affect the character of the work to be done, the treatment of the pier would remain the same. Furthermore, as you must be aware, Pier 4 is surrounded with rip-rap, and in sinking our caissons around that pier we went through a solid mass of rip-rap stone nearly all the way down; consequently, it will be practically impossible to get a rod or a hook down the side of the old crib, so as to reach the cutting edge of the old caisson underneath it. This I believe was finally accomplished with some uncertainty at Pier 2, but in my judgment it will be entirely impossible at Pier 4, and it will be very foolish, after the coffer-dam is pumped out and we are ready to concrete, to waste any time or assume any risk in any such idle experiments. You may safely take it for granted, that wherever the cutting edge of that old pier is, the pier is settling and leaning dangerously, and must have the support which we propose putting in there, and that just as quickly as possible when the water is pumped out.

"Answering your question directly as to hunting for this cutting edge, will say that, of course, if your company will assume all responsibility and all costs, direct and indirect, and all consequent damages, which might occur from floods and other disasters, by delaying the putting in of that concrete around Pier 4, after we are ready for it, then, of course, we will hunt for the cutting edge of that caisson, at your expense, just as long as you want to continue such investigations, but when you think the matter over carefully, I think you will agree with me that no time should be lost, after we are ready to concrete, for the purpose of satisfying curiosity. I am thoroughly satisfied myself as to where the cutting edge is under that old pier, and when you see the top of the old crib, you can satisfy yourself beyond a doubt as to where the bottom is.

"Very truly yours,

"A. J. TULLOCK,
"Proprietor."

APPENDIX D

LETTERS OF RALPH MODJESKI CONTAINING RECOMMENDATIONS FOR REINFORCEMENT OF THE PIERS.

"CHICAGO, ILL., August 5, 1911.

"MR. E. J. PEARSON,
"*First Vice-President,*
"Missouri Pacific Railway Company,
"St. Louis, Missouri.

"DEAR MR. PEARSON:—As had been prearranged, I visited, last Thursday, in company with Mr. C. E. Smith, Bridge Engineer, your two bridges at Little Rock, Arkansas. Most of the time was devoted to the examination of your Little Rock Junction Bridge, which is in a precarious condition. I have also carefully examined the plans and records of the old structure, together with what has been done to repair it.

"It would seem that the coffer-dams which were placed by pneumatic process around Piers II and IV did not get at the seat of the trouble, and as shown by Pier IV did not correct it. The original caissons were sunk to rock and the air chamber filled with concrete, but the cribwork above was filled with sand and possibly some rip-rap. There is no doubt in my mind that the concrete in the caisson chamber rests on rock. It would be difficult to suppose that the engineers had come within a foot or two from rock and had failed to remove that thickness of material before sealing the chamber. The records as well as the tradition seem to indicate positively that the air chambers were sealed with concrete. There seems to be no doubt therefore that the trouble occurs in the cribwork above, which, instead of being filled with concrete as it should have been, was filled with sand and possibly some rip-rap. This sand is probably leaking out through the openings in the outside sheeting and is settling away from the timbers, so that the entire load is carried on those timbers, which under a load of something like four or five hundred pounds per square inch are crushing.

"I should like to have one or two more days to consider this whole matter carefully, and will therefore not make this my final report as to what is best to be done. I have no doubt, however, that what we should attempt to do is to remove the sand from the inside of the cribs and replace it with concrete. If that is done to all the piers it would afford excellent foundations for future piers which you may wish to construct for a new bridge or else it would support the present piers as long as the piers themselves held together.

"Pier III is the one which should receive immediate attention as it is gradually settling at a rate which in a few months time

would mean its total destruction. In conference with Mr. Smith it was agreed that a plant be rigged up on a barge for taking wash-borings around the pier to find out if there is sufficient rip-rap present to prevent driving a wooden triple-lap sheet-piling coffer-dam. It is also desirable to arrange as soon as possible to bore at least two holes in the up-stream corners of the coffer-dam to find just how much material is left inside of it, and also incidentally to find whether the working chamber has actually been sealed with concrete.

"My present plan would be to drive a coffer-dam around the pier at a distance of say 6 ft., from the walls of the old crib and to pump it out, lowering the water level sufficiently to examine the old crib by partly removing the outside planking. While this is being done it would be quite advisable to partly support the up-stream end of Pier 3 by shores which in their turn would rest on piles driven alongside the old coffer-dam. Mr. Smith is familiar with what I have in mind.

"Whatever work is done, it will have to be performed carefully and step by step, in order not to endanger the present pier. It is not possible to foresee all the difficulties that may be encountered, but I am quite confident that the work can be done at a reasonable cost.

"I would recommend that the work be done by Company forces, thus saving considerable time and possibly expense as well.

"I expect to write you again Monday, after studying the situation a little further.

"Very truly yours,

"RALPH MODJESKI."

"CHICAGO, August 7th, 1911.

"MR. C. E. SMITH,
"*Bridge Engineer,*
"Missouri Pacific Railway,
"St. Louis, Mo.

"DEAR MR. SMITH.—I have been thinking a great deal about the Little Rock proposition since I saw you at St. Louis, and I cannot arrive at any more satisfactory solution than the one which I outlined. There is another possibility, however, which occurred to me, and that is that it may not be necessary to drive a coffer-dam around the pier at 6 ft. distance from the old coffer-dam. The farther away we go the more difficult it would be to keep the water out during pumping. I would suggest, therefore, that, as contemplated, you take the soundings and borings to ascertain how much rip-rap there is around the pier, and then either simultaneously or immediately thereafter arrange to bore a hole through the corners of the cribs vertically to find out how much sand or rip-rap or other material there is in the crib. It may be that if the material contained in these

cribs is merely sand or such material as can be pumped out, it will be sufficient to drive a Wakefield sheet-pile coffer-dam at a distance of 1 ft., for instance, in which case it would be very easy to excavate the material between the old crib and the coffer-dam and seal the bottom with concrete. It would take very little concrete and of little thickness to hold water. After this is done we could pump out the coffer-dam sufficiently to get at the top timbers of the crib. We could then remove these timbers at least partly and remove as much as possible of the material inside. It is probable that we might be able to pump out the water from the interior of the crib and then fill it with concrete in the dry.

"Very truly yours,

"RALPH MODJESKI."

"CHICAGO, August 19, 1911.

"MR. E. J. PEARSON,

"Vice-President, Missouri Pacific Railway,

"St. Louis, Mo.

"DEAR MR. PEARSON: Since writing you the report on the Little Rock Junction Bridge, I have been thinking the matter over, and it occurs to me that if the Company desires to maintain this bridge for some years to come and perhaps build a new superstructure on it, it would be cheaper in the end to rebuild the piers of concrete, at least Piers 3 and 4, which are causing the most trouble. Pier 3 is in a precarious condition, and no method of repairing, unless it be a very expensive one, can be counted on with absolute certainty to produce satisfactory results, and at best the masonry in the present piers is not in the very best condition, many of the stones being cracked, and while it may last for many years, it would have to be replaced some time or other. I would respectfully suggest that you authorize the reconstruction of Piers 3 and 4 at this time. In this way the masonry can be taken down and the cribs filled solid with concrete at much less expense than could be done with the piers in place. The concrete piers could be built for a reasonable amount and part of that cost would be offset by the saving in the cost of the foundation repairs, which repairs, as stated, would be rendered considerably easier.

"Kindly advise me what your wishes are in this respect.

"Very truly yours,

"RALPH MODJESKI."

APPENDIX E

List of Equipment Used in Repairs to Pier 3.

1 barge, 24 by 80 ft. with derrick.
1 barge, 18 by 60 ft.
1 barge, 16 by 50 ft.
1 barge, 15 by 24 ft.
1 rowboat.
1 compressor, 18 by 18¼ by 24 in., with receiver and mains.
2 air locks.
2 hoisting engines.
3 horizontal boilers.
1 derrick.
1 orange-peel dredge, ¾-yd.
1 pile leads and hammer (not used).
1 No. 4 Emerson pump.
1 pulsometer.
1 12 by 7 by 12 Fairbanks-Morse pump.
1 D. C. centrifugal pump, 8-in.
2 steam jets.
1 diaphragm pump.
1 double-drum crab.
1 No. 11 Smith mixer.
4 steel concrete cars.
1 turntable.
6 steel concrete buckets.
1 steel tremie.

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PROOF OF AN ASSUMPTION IN THE THEORY OF CONCRETE BEAMS.*

By Ralph E. Goodwin, Jun. Am. Soc. C. E.

Synopsis.

The object of this paper is to present a mathematical justification of the theory that a section of a beam which is a plane before bending remains a plane after bending, as applied to beams of reinforced concrete.

The proof is based on the principle of least work.

The assumptions are those usual in the theory of reinforced concrete design, with the exception of that of a plane before and after bending, which is assumed to be true of the compression side of the beam, but not of the beam as a whole.

It is important to notice that the value of the conclusions depends on the accuracy of the assumptions. No assumptions are made in the proof, however, which are not commonly used in reinforced concrete design.

The common theory of reinforced concrete beam design is based on the assumption that a section which is a plane before bending remains a plane after bending. This assumption is borrowed from the common theory of flexure, and is so familiar to engineers, through its application to beams of homogeneous material, that the correctness of applying

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

it to beams of reinforced concrete is seldom questioned. Nevertheless, the difference between a beam of homogeneous material and a composite beam of concrete reinforced with steel is so great that an assumption which is reasonable in the case of the former is not necessarily reasonable in the case of the latter, and textbooks on the subject should furnish more plausible reasons for the theory of a plane before and after bending than they usually advance.

There follows a mathematical justification of the principle in question, which is based on the principle of least work. It is assumed that the correct value of j (arm of resisting couple) is the value which develops any given M (resisting moment) with the minimum of work expended in bending the beam. The method of proof used is to compute the value of j by the method of least work and to show that the value thus determined is the same as that in common use.

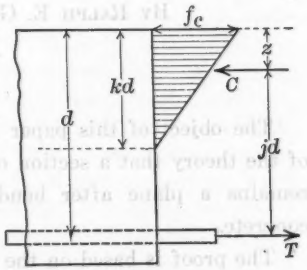

FIG. 1.

In the work which follows, the assumptions usual to the theory of reinforced concrete design are made, with the exception of that of a plane before and after bending, which is assumed to be true of the compression side of the beam, but not of the beam as a whole (Fig. 1).

The symbols used are the standard notation. In addition,

dl = the distance between two adjacent cross-sections of the beam before bending.

W_{dl} = the work done in straining a section of the beam of length, dl.

W_{dl} = (work of straining steel on tension side) plus (work of straining concrete on compression side).

According to the principles of mechanics, the work done by a load gradually applied is one-half the force multiplied by the distance.

Hence,

$$W_{dl} = \frac{1}{2} C \frac{2}{3} \frac{f_c}{E_c} \times dl + \frac{1}{2} T \frac{f_s}{E_s} \times dl$$

$$= \frac{1}{2} \left(\frac{f_c b k d}{2} \right) \frac{2}{3} \frac{f_c}{E_c} \times dl + \frac{1}{2} (A f_s) \frac{f_s}{E_s} \times dl$$

$$W_{dl} = \frac{f_c^2\, b\, k\, d}{6\, E_c} \times dl + \frac{f_s^2\, A}{2\, E_s} \times dl \qquad (1)$$

$$M = \frac{f_c\, b\, k\, d}{2} \times j\, d = f_s\, A\, j\, d\,;$$

$$f_c = \frac{2\, M}{b\, k\, j\, d^2} = \frac{2\, M}{3\, b\, (1-j)\, j\, d^2}; \quad f_s = \frac{M}{A\, j\, d}.$$

Substitute these values in Equation (1).

$$W_{dl} = \left(\frac{2\, M}{3\, b\, (1-j)\, j\, d^2}\right)^2 \frac{3\, b\, (1-j)\, d}{6\, E_c} \times dl + \frac{M^2}{2\, A\, j^2\, d^2\, E_s} \times dl$$

$$= \frac{M^2}{2\, d^3\, b\, E_c} \left\{ \frac{4}{9\,(1-j)\, j^2} + \frac{1}{p\, n\, j^2} \right\} dl$$

$$= \frac{M^2}{2\, d^3\, b\, E_c} \left\{ \frac{4\,(j^2 - j^3)^{-1}}{9} + \frac{j^{-2}}{p\, n} \right\} dl.$$

To find the value of j which will make W_{dl} a minimum, set the first derivative with respect to j equal to zero.

$$\frac{d\,(W_{dl})}{d\,j} = 0 = \frac{M^2}{2\, d^3\, b\, E_c} \left\{ \frac{-4\,(j^2 - j^3)^{-2}\,(2\,j - 3\,j^2)}{9} - \frac{2\, j^{-3}}{p\, n} \right\} dl;$$

$$0 = \frac{4\,(2\,j - 3\,j^2)}{9\,(j^2 - j^3)^2} + \frac{2}{p\, n\, j^3};$$

$$0 = \frac{1}{j^3\,(1-j)^2} \{ 4\, p\, n - 6\, j\, p\, n + 9\,(1-j)^2 \}.$$

Each factor, when set equal to zero, gives a root of the equation. The root desired lies in the last factor,

$$0 = 4\, p\, n - 6\, j\, p\, n + 9\,(1-j)^2;$$

$$0 = j^2 - \frac{2}{3}\, j\,(p\, n + 3) + \left(1 + \frac{4}{9}\, p\, n\right).$$

Solving for j by completing the square in the above equation gives,

$$j = 1 - \frac{1}{3}\left(\sqrt{2\, p\, n + (p\, n)^2} - p\, n\right) = 1 - \frac{1}{3}\, k.$$

Hence $$k = \sqrt{2\, p\, n + (p\, n)^2} - p\, n.$$

This is the value of k in common use. *Q. E. D.*

For beams reinforced for compression and for T-beams the demonstration is similar to the foregoing, up to and including the placing

of the first derivative equal to zero. From that point the general solution becomes so complicated that it is best to continue as follows,

(1) Assume numerical values for p, p', d, d', n;

(2) Compute the numerical value of k from the formula in common use;

(3) Substitute the foregoing values in the following equation:

First derivative of work with respect to $k = 0$.

It will be found that the equation in (3) is always satisfied by the values assumed in (1) and (2). *Q. E. D.*

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DEPRECIATION AS AN ELEMENT FOR CONSIDERATION IN THE APPRAISAL OF PUBLIC SERVICE PROPERTIES.*

By C. E. Grunsky, M. Am. Soc. C. E.

Synopsis.

The object of this paper is to bring nearer to standardization the methods of dealing with depreciation when valuations are made for rate-fixing purposes; to show that the repayment to the owner of invested capital has nothing to do with depreciation; to show that the investment should not be returned to the owner unless he is to be bought out; to show that a public utility taken as a whole may be regarded as having unlimited life; to show that in fixing rates the replacement requirement may with advantage be allowed to take the place of the depreciation allowance; to show that departure of actual from probable life should be taken into account in estimating replacement requirements or depreciation; and to show that the method here called the Unlimited-Life Method of making appraisals is better suited than other methods, to serve as the basis for estimating necessary earnings.

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

All methods of valuing public service properties for rate-fixing purposes—to be fair to both owner and rate-payer—should be in continuous application from the beginning of operation.

It follows from this that the past history of expenditures and earnings should be given consideration and that it may be unfair to assume that, because of depreciation in fact, a part of the investment has been amortized and that if not amortized the fault is the owner's. (Knoxville case.)

The invested capital should remain unimpaired. The owner's earnings should include whatever may be necessary to meet maintenance, repairs, and replacements, but no part of his investment should be returned to him, unless the acquisition of the property is involved.

The situation is not altered even in those cases in which the owner diverts to outside uses the funds which he has earned to meet replacements. The owner's obligation to replace the worn-out parts remains, and must be met if the rate-payer is to be served satisfactorily.

Amortization of the invested capital becomes necessary only when the public service property is to be acquired by the public.

The allowance in the earnings which is usually made to the owner under the name of depreciation, whether applied to this use or not, is really intended as an allowance to meet replacements.

Replacement requirements can be determined from the cost to replace and the time when the replacement will probably take place.

The time when an article which has already been in use for some time—which, in other words, has acquired age—will go out of service cannot be determined by subtracting the age from the probable life of a new similar article. It is dependent on the condition of the article. It is approximated from the article's expectancy—from its remaining probable life. Expectancy changes with age.

When expectancy is given due weight, the application of the Equal-Annual-Payment and other similar methods of valuation becomes difficult if not impracticable.

Expectancy is capable of much closer approximation than by the subtraction of age from probable life.

Taken as a whole, each public service property may be regarded as having unlimited life. The probable life of its parts—whether only a railroad tie or an electric generator—does not alter this unlimited life

of the whole. The expectancy of each part, however, may be used in estimating the "owner's obligation to replace," which is another name for what is usually called "accrued depreciation".

The obligation to replace each part, expressed in dollars, grows with the age of the part. If the replacement requirement is properly estimated and covered by the earnings, the owner can keep the plant at 100% efficiency, based on the standard established by the plant itself; but such replacement allowance cannot be regarded as a return of capital until the time comes when the owner is relieved of any further obligation to replace.

The rate of depreciation is an aid in making an advance determination of the replacement requirements, and to this extent is an element to be considered in fixing rates.

The accrued depreciation, or the accrued obligation to replace discarded or inefficient articles, becomes of interest when for any reason the remaining value of physical elements is to be ascertained.

Conclusion.—The Courts are in error when they hold it to be always necessary to give depreciated value consideration when rates are to be fixed.

The fair application of any method of valuation to a property which has acquired age, when this valuation is to be used as a basis for fixing service rates, involves due consideration of past history, particularly in so far as the method is concerned by which necessary earnings in the past were estimated.

Continuity of application is essential to the fairness of the various methods of making appraisals for rate-fixing purposes.

No part of the capital properly invested in a public service property should be returned to the owner until a transfer of ownership to the public is contemplated.

The appraiser should concern himself with the replacement requirements, rather than with amortization of capital or with depreciation.

In estimating depreciation, accrued depreciation, or replacement requirements, the expectancy, properly determined, should be substituted for probable life new less age.

All public service properties should be regarded as having an unlimited life, and, if thus regarded, a new method of valuation becomes possible in which operating expenses are made to include the

replacement requirements and under which there will be no amortization of capital.

The Unlimited-Life Method of valuing public service properties has the great advantage of requiring the smallest earnings in the early years of the property's life; it is simpler than all others in its application; and it is fair to both owner and rate-payer.

Despite the tendency which is apparent on the part of the Courts and the public service commissions, and especially on the part of the lawyers and experts who represent the rate-payers, to favor appraisals based on cost of reproduction, less depreciation, as a vital element to be considered in fixing rates, there is no doubt that there will be a backward swing of the pendulum, and that the fundamental principle will ultimately be recognized, under which the owner of a public service property will be regarded substantially as the agent of the rate-payer; and, as such, will be allowed, and may even be required, to keep his investment in the property unimpaired and undiminished, just as would be the case if the rate-payers themselves, through the agency of a municipality or otherwise, had constructed the public service plant. This is good policy, sound in every way, and, when well understood, being absolutely fair to all concerned, providing as it does for the most reasonable apportionment of earnings to various periods of a plant's life, should, in all cases in which it is applicable, prove acceptable alike to the rate-payer and to the owner.

Some of the public service commissions of the country, as well as the Courts, are confounding "amortization" with "depreciation". The result is unfortunate, and has led valuation experts to adopt all sorts of subterfuges to secure for their clients that equity to which, whether rate-payers or owners, they are entitled.

It has become common practice, nevertheless, frequently an improper practice, to regard as a repayment of capital all so-called "depreciation" which has been allowed in the earnings (and, let it be assumed, which has actually been collected from the rate-payer).

Any repayment of capital is "amortization", and has nothing whatever to do with the depreciating service value of any part of a public service plant.

The repayment of capital (amortization), which it has become customary to assume is taking place when earnings include an allow-

ance for depreciation, is an unfair burden on the rate-payer in the early years of operation.

This can be and is avoided under the rational system of making suitable provision for the replacement of worn-out parts of the property, instead of endeavoring to amortize the investment.

To make this clear and to point out some of the errors into which Courts and some experts are drifting is the main purpose of this paper. Incidentally, a fact which is too often lost sight of will be emphasized, namely, that the various methods of computing the current depreciation allowance which should be covered by the earnings are only then correct when applied consistently and continuously throughout the probable, or, as the case may be, the useful, life of an article; that, therefore, the dictum of the Courts, according to which no regard should be paid to the fact that past earnings may have been insufficient to provide a depreciation fund, is at fault, and should be suitably modified or at least qualified.

The writer will endeavor, moreover, to show that appraisals for rate-fixing purposes may always be made at the full amount of the investment and that, when applicable, the method of computing and allowing replacement requirements under the assumption that the property as a whole has unlimited life, instead of allowing depreciation as ordinarily defined, is the one which should find general acceptance.

Fair Value.—The term "fair value" as used in connection with rate-fixing is difficult to define. This term is used in a measure, no doubt, for the purpose of ruling out "book value" or actual investment, which may include items and amounts of doubtful propriety, and to rule out values based on stock and bond issues and the market values of such securities. The term has not yet been defined satisfactorily, and no attempt will be made here to reconcile divergent views in relation thereto; but attention may be called to the difficulty which has been experienced by all who have attempted to make appraisals for rate-fixing purposes, in reconciling "fair value" to a purchaser with the "fair value" which is to be considered when fixing rates. Why should there be one valuation for purchase and another for rate-fixing? The answer that has been given by the Courts (Knoxville case) is practically to the effect that there should be no such difference, and experts have been put to it to determine values so that any such difference shall disappear.

The value to an investor is unhesitatingly determined from the net earnings, with due regard to hazards of the business. The value for rate-fixing purposes is to be that value on which, with the same regard for the hazards of the business, the owner is to be allowed to earn a fair interest return. Value should be the same whether determined by a rate-fixing body or whether determined for a purchaser. "Fair Value", then, is necessarily based on proper and reasonable investment which may be ascertained from cost records, and when cost records are not available, is usually estimated by the "cost-of-reproduction" method. Any determination of value by comparison with the cost of the next best substitute for the established utility is at times unwarranted and absurd, and has no place in determining the equities between the owner and the public whom he is serving, beyond establishing an upper limit of possible value.

"Fair Value" may include certain more or less indeterminate elements, such as the cost of establishing the business, including early deficient earnings; but no definite or precise rules can be laid down for determining these, as it would be eminently unfair to reward the least successful enterprise which shows greatest early losses and highest unproductive expenditures, by granting it the greatest allowance for so-called "going concern" or "establishment of business" or other similar values.

"Investment" or "Capital Invested."—This is the aggregate of the reasonable and proper expenditures which have been made to render the property in question efficient for the purpose for which it is intended.

"Depreciation."—This is the lessening in worth of any perishable article or property. It is not, therefore, to be measured by inherent deterioration, due to wear and tear, but solely by a comparison between the probable life, the expectancy, and the cost of replacement at the expiration of the term of the article's actual serviceability.

"Annual Depreciation" or the "Annual Depreciation Increment."—This is the annual theoretical lessening in worth, expressed in dollars.

"Accrued Depreciation."—This is the difference between the original cost and the "remaining value".

"Remaining Value" or "Present Value."—This is the present worth of any article, dependent on, and to be computed from, the three elements: (*a*) the probable useful life of the article new; (*b*) its probable remaining life; and (*c*) the cost of replacement, which, when prices

are not subject to change, is the original cost less the residual value of the article when it goes out of use.

"Residual Value."—This is the value which remains in any article after it has ceased to be useful as an integral part of the public service property. It may be scrap value, or it may be the price at which it is disposed of for use in connection with some other property. It is usually estimated as the value to an outside purchaser, less the cost of delivery to such purchaser.

"Unrecovered Depreciation" or "Uncollected Replacement Annuities."—This refers to the allowed depreciation increment or to the allowed replacement annuity which the earnings are inadequate to cover, and which, in consequence thereof, could not be collected.

"Probable Life."—This is the probable time, usually expressed in full years, during which any article will render efficient service. This is to be determined from the sum of experiences with like articles and with due regard to local conditions. It depends not alone on the time required for an article to deteriorate and become valueless by use, by ordinary wear and tear, but also on the time when, by reason of accidental destruction, inadequacy, or obsolescence, the article must be replaced by a new one better adapted to fulfill its purpose.

"Remaining Life" or "Expectancy."—This is the probable time, expressed in years, during which any article which is no longer new will continue to render efficient service. The remaining life cannot always be determined from the probable life of the article when new by subtracting therefrom its age. The condition of the article and the local circumstances should be taken into account in estimating the remaining life.

A high-duty pump, which originally had a probable life of 25 years, when it reaches the end of this term, may be in a condition almost as good as new. It has escaped the possible accidents of the early years of its life, and by careful attention and replacements of its wearing parts is still rendering first-class service. The value of this pump is not to be written off the books, neither should it be regarded as good as new. Its value is ascertained by determining its probable additional years of usefulness and the probable cost of replacing it at the end of the new term.

An irrigation canal usually improves with age. So far as wear and tear is concerned, it has unlimited life; but, under the develop-

ment of extensive areas, the small original canal may in the course of time be superseded. The probable life of this canal, and therefore, too, the annual replacement increment, is estimated on some assumption relating to the rate of this development. Finally the time comes when the project for a comprehensive canal system has taken definite shape and it may reasonably be assumed that within a definite period, five or ten, or some other number of years, the original canal will be superseded; its diverting dam and its head-works, and perhaps the canal itself, will then be abandoned. The remaining life or expectancy of the canal is at that time only five or ten, or some other number of years, as the case may be, and within this time the remaining investment in or the value of the canal is the amount under consideration for replacement.

"Amortization."—In the sense in which this term is used in this paper, it applies to the retirement of capital. Ordinarily, the discarded article is replaced by a new one which will render equivalent or better service. The capital to be amortized during the life of the article is, for all practical purposes, its replacement cost, being the cost of installing an equivalent new article, less the residual value of the original article.

"Annual Replacement Increment" or "Replacement Annuity."—These are terms used to designate an annual allowance or an annual contribution to a replacement fund, such that this annual allowance, together with interest thereon (at the rate of the net earnings of the property) will, during the life of any perishable article, amount to its replacement cost.

"The Obligation to Replace."—This is an obligation, carried by every owner of public service property, to replace articles forming essential parts of public service plants when worn out or no longer capable of rendering adequate service. The "Obligation to Replace", when expressed in dollars, is equivalent to the accrued theoretical depreciation or to the sum which should have been earned and accumulated to meet necessary replacements. This obligation to replace, together with deferred maintenance, must always be considered by the appraiser of property which is to be purchased. The investment less this obligation to replace (also less deferred maintenance) is, in other words, a measure of present worth.

"Deferred Maintenance."—This is the neglect, expressed in dollars, which has resulted from failure to keep an article in good condition and repair. It is the sum which should at once be expended to restore the article to ordinary good service condition, and to protect it in such a way against causes of rapid destruction that its deterioration will not be unduly rapid. Ordinarily, there should be no "deferred maintenance." By proper attention to maintenance and repairs, the service rendered should be up to the standard which is expected of the plant. In other words, the plant should, practically at all times, be at 100% efficiency.

"Wear and Tear."—This is the term applied to the deterioration of an article from use. The article is kept in serviceable condition by the renewal of its worn-out parts and by suitable attention to its wearing parts. Maintenance and repair are to be included in operating expenses, and are not covered by depreciation in the sense in which used in this paper. The expenditures for repairs and maintenance are intended to keep the article at or near 100% efficiency. Repair and maintenance expenses usually increase somewhat with the age of an article, and, when this increase is worthy of note, may be an element to be taken into account in determining present worth.

"Replacement Cost."—This is the cost of the article with which a worn-out article is replaced, less the residual value of the original article. It is the cost of effecting a change from the worn-out part to a new part of equal service value. This is sometimes called "wearing value."

"The Sinking-Fund Method."—This method of making appraisals and of determining necessary earnings is one under which the annual allowance for depreciation is uniform in amount. This depreciation allowance is the annuity which, together with interest at the rate of the net earnings of the property, will, during the probable life of an article, amount to its replacement cost. The replacement, it is assumed, is to be accomplished at the end of the probable life of that article. There is no repayment of capital. The investment remains undiminished. This, therefore, is a 100% valuation method and might be thus designated. It is the method referred to by the writer as Method No. 1, in a former paper* on appraisals.

* "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 828.

The "Equal-Annual-Payment Method", as described in the report of the Special Committee on Valuation, as presented to the Annual Meeting of January 21st, 1914, refers to that method which makes the annual depreciation, or amortization increment, an amount increasing from year to year. The annual depreciation is estimated by sinking-fund methods. It is, for any year, equal to the annuity which will retire the remaining value in the remnant of the original probable life term and, uniformity of interest rates being assumed, it will, when added to the interest on the remaining value, always amount to the same sum. The application of this method is exactly equivalent to the application of the Sinking-Fund Method.* When depreciation allowances computed under this method are actually earned from the time an article goes into service, they may be considered as refunds of invested capital. The invested capital, in that case, decreases as depreciation; if thus earned, it accrues. Depreciation earnings may properly be regarded as being thus applied to retired capital, but this method of estimating what should thus be earned from year to year is undesirable, because it involves frequent, elaborate, and cumbersome re-estimates of remaining value, requiring the services of experts, although it is absolutely identical in results with other methods which have the advantage of simplicity. Furthermore, when not applied from the beginning, there may never be any surplus in such an amortization account for application to a refund of early investments and this portion of the investment may remain undiminished.

"The Straight-Line Method."—This "method of estimating the annual depreciation", or "the annual amortization installment", makes the annual amortization of capital uniform throughout the probable life of an article. The annual amortization installment is estimated from the original cost of an article and its probable life, by dividing the cost by the number of years. This, like any other amortization method, is justified on the theory of the immediate application of an annual installment to the retirement of capital. In this respect there is an essential difference between this and the Sinking-Fund Method, under which no capital is retired, and also between this and the Unlimited-Life Method, under which, also, there is no retirement whatever of the investment.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 827–829–834–842, etc.

"The Unlimited-Life Method."—This method of making appraisals which are to serve as a basis for fixing rates, has been referred to in the writer's paper on appraisals.* In that paper the term "perpetual life" was used. It is believed to be more appropriate to use the designation here suggested.

Nearly all public service properties, taken in their entirety, may be regarded as having unlimited life. There is then no accrued depreciation to be deducted in making the appraisal. The property is kept in good condition by the repair of its parts and by replacements as these parts become useless and have to be discarded. No part of the investment is returned to the owner, but, as the plant grows old and one part after another has to be replaced, he must be allowed to recover in the earnings the cost of each article as replaced. It is on this principle that many complex, and particularly public service, properties are operated. In the early years of the life of a property made up of many parts, such as rails and ties, the replacement requirements will be small. As the property acquires age, the replacements —provided that extensions are relatively unimportant and negligible in comparison with the extent of the property under consideration— will gradually increase to nearly the amount indicated as amortization by the Straight-Line Method. The departure from this amount will be dependent on the annual extensions of the system in relation to the entire investment. There will be ultimate agreement between the replacement requirement and the straight-line amortization increment, if the plant is one that has ceased to grow. The valuation for rate-fixing purposes will always be at 100%, and the replacement requirements, until definitely ascertained by experience, will be approximated with due consideration of the age and the probable life new of the individual parts of which the property is made up.

Diagrammatic Illustrations.

The foregoing definitions, relating to depreciation, to replacement annuities, to the obligation to replace, and to amortization, can best be made clear by reference to diagrams Figs. 1 to 5.

These diagrams, being for the purposes of illustration, apply to the special case in which the article under consideration has no residual or scrap value at the time of its replacement, and in which the new

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 805, 815, and 875.

article which replaces it costs the same as the original article and has the same service value.

Referring to Fig. 1, illustrating the Straight-Line Method: $A\ B$ is the original investment of capital in an article, subject to depreciation. $I\ J$ is the portion of the original investment which has been amortized (sometimes regarded as the accrued depreciation). $K\ L$ is the annual amortization increment. $H\ I$ is the remaining investment. $A'\ B' = A\ B$ is the replacement cost. $B\ A'\ B'\ A''\ B''$, etc., is the investment line showing for any particular time the amount of capital still invested in the article under consideration. $M\ N$, in the earnings diagram of Fig. 1, represents the interest on the remaining value. $N\ O = K\ L$ represents the annual amortization.

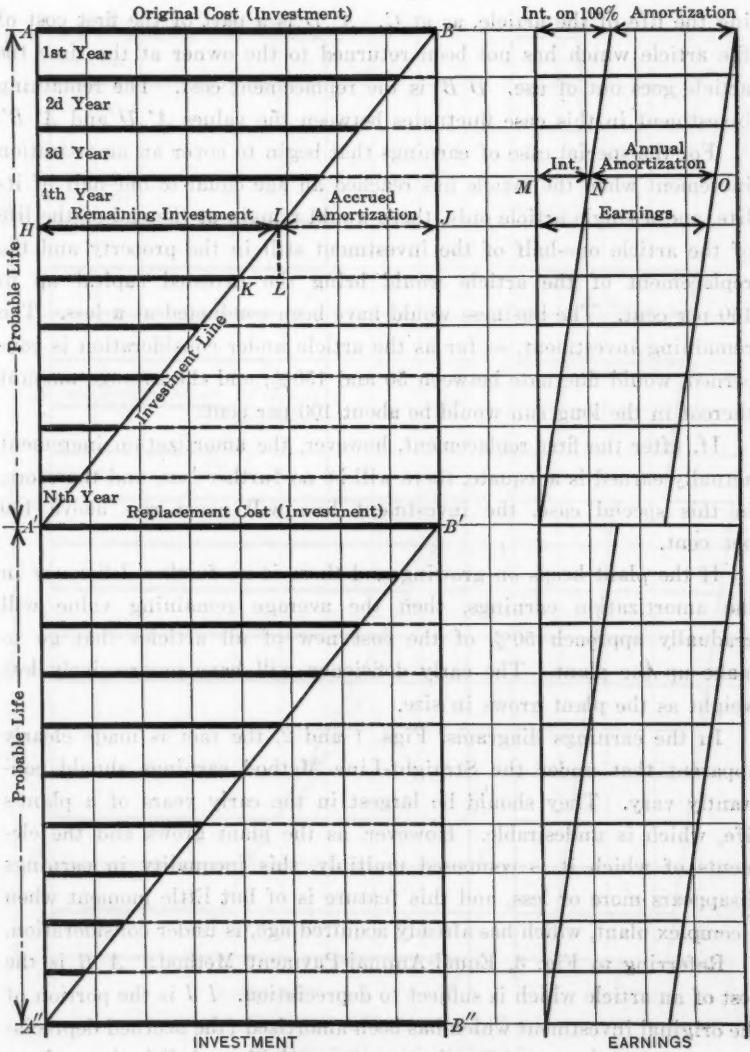
The owner is entitled to a reasonable return on the remaining investment, also to the straight-line amortization increment $K\ L = N\ O$, besides, of course, operating expenses, including taxes, insurance, etc.

The fluctuations in the amount of capital remaining in the property are shown by the "investment line." They range from 100% for the article new to nothing at the end of its term of life. The capital returned to the owner, whether under the name of "capital amortization" or under the name of "depreciation", is his, to do with as he pleases. It is immaterial whether he places it in a special depreciation fund; whether he makes outside investments therewith; or whether, as the property develops, he reinvests it in the extensions thereof, provided, of course, that at the end of the life of the article he is prepared to reinvest the capital required to replace it. If correctly estimated and earned from the beginning, the accumulation in the fund (without interest thereon, because such interest must be regarded as outside earnings) will be just adequate to replace the worn-out article.

Under this method of amortizing capital, the earnings must be somewhat larger in the early years of the article's life than in the later ones. This is an objectionable feature.

For an infinite number of such articles, of all possible ages, the aggregate remaining investment will be 50% and the straight-line amortization increment will be just equal to the annual replacement requirement.

For a single article, in case the earning of the amortization increment does not begin until some years have elapsed, the investment line will take the shape of the broken line in Fig. 2.



STRAIGHT-LINE AMORTIZATION

FIG. 1.

Referring to Fig. 2, Incomplete Amortization Straight-Line Method: Amortization earnings are supposed to commence at some time during the life of the article, as at C. $A'\ D$ is a part of the first cost of the article which has not been returned to the owner at the time the article goes out of use. $D\ B'$ is the replacement cost. The remaining investment in this case fluctuates between the values $A'\ D$ and $A'\ B'$.

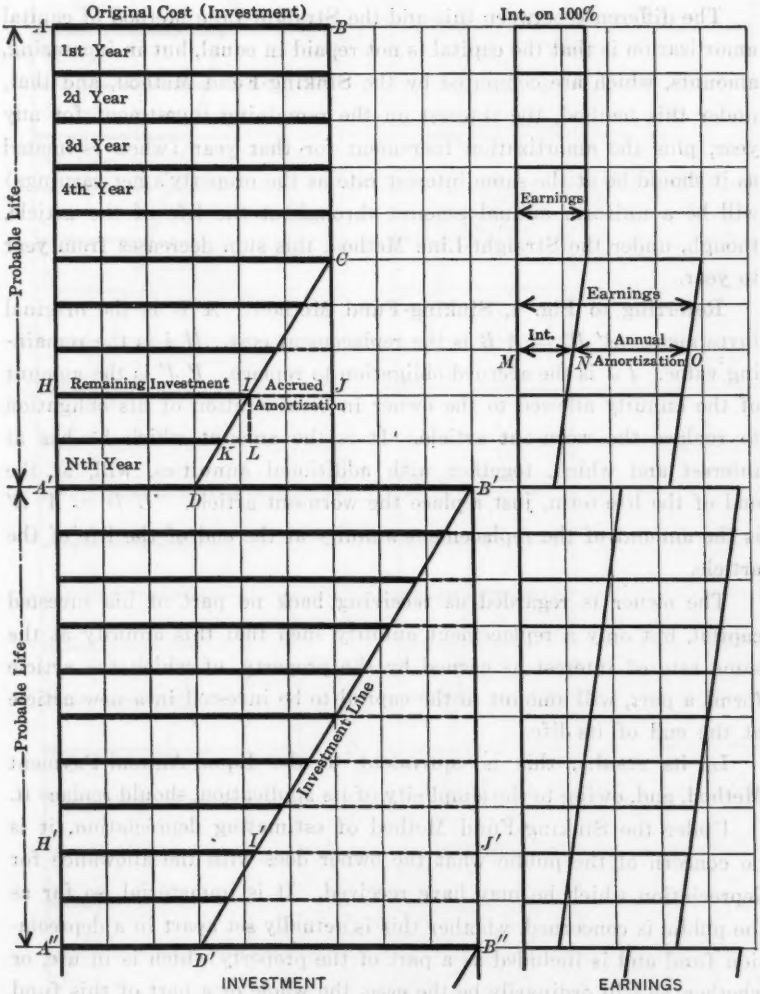
For the special case of earnings that begin to cover an amortization increment when the article has reached an age equal to one-half of its life, and a single article only, there would remain at the end of the life of the article one-half of the investment still in the property and the replacement of the article would bring the invested capital up to 150 per cent. The business would have been conducted at a loss. The remaining investment, so far as the article under consideration is concerned, would fluctuate between 50 and 150%, and the average amount thereof in the long run would be about 100 per cent.

If, after the first replacement, however, the amortization increment actually earned is adequate, there will be no further loss, and therefore, in this special case, the investment line will never rise above 150 per cent.

If the plant keeps on growing and there is no further deficiency in the amortization earnings, then the average remaining value will gradually approach 50% of the cost new of all articles that go to make up the plant. The early deficiency will have progressively less weight as the plant grows in size.

In the earnings diagrams, Figs. 1 and 2, the fact is made clearly apparent that under the Straight-Line Method earnings should constantly vary. They should be largest in the early years of a plant's life, which is undesirable. However, as the plant grows and the elements of which it is composed multiply, this inequality in earnings disappears more or less, and this feature is of but little moment when a complex plant, which has already acquired age, is under consideration.

Referring to Fig. 3, Equal-Annual-Payment Method: $A\ B$ is the cost of an article which is subject to depreciation. $I\ J$ is the portion of the original investment which has been amortized (the accrued depreciation). $H\ I$ is the remaining investment. $A'\ B' = A\ B$ is the replacement cost. $B\ A'\ B'\ A''$, etc., is the investment line representing for any particular time the amount of capital invested in the article under consideration. $M\ N$, in the earnings diagram of Fig. 3, represents the



INCOMPLETE AMORTIZATION
STRAIGHT-LINE METHOD.

FIG. 2.

interest on the remaining investment. $N\ O$ represents the annual amortization. $M\ N + N\ O$ is a constant.

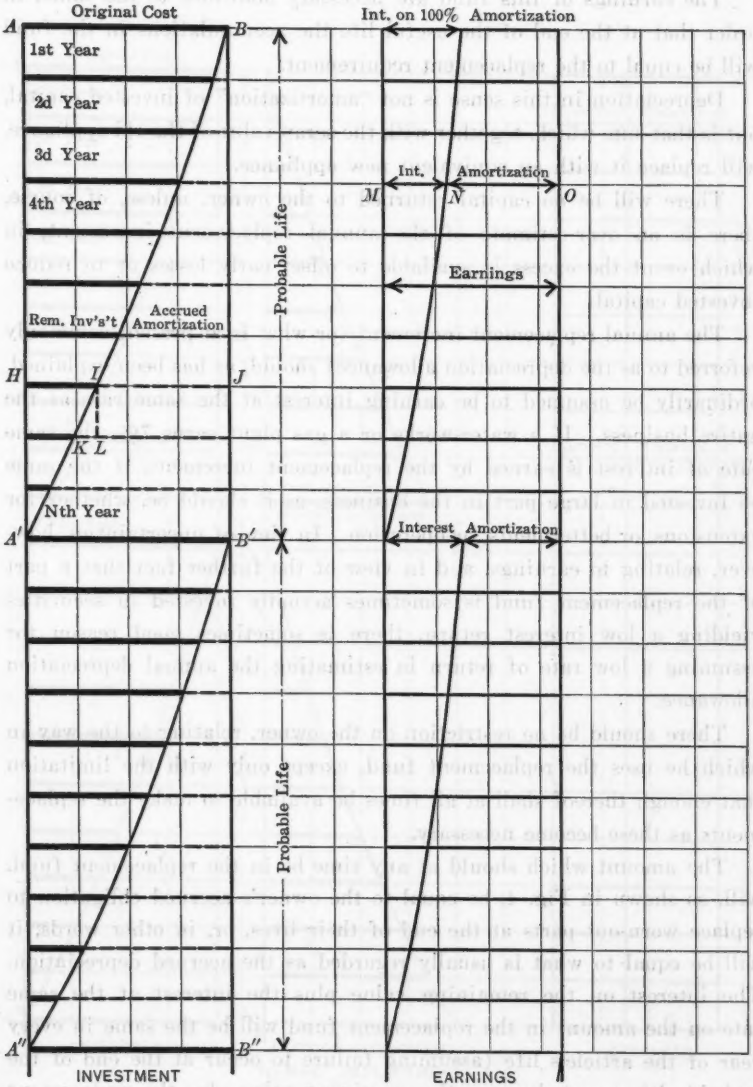
The difference between this and the Straight-Line Method of capital amortization is that the capital is not repaid in equal, but in increasing, amounts, which are computed by the Sinking-Fund Method, and that, under this method, the interest on the remaining investment for any year, plus the amortization increment for that year (when estimated as it should be at the same interest rate as the property's net earnings) will be a uniform annual amount throughout the life of the article, though, under the Straight-Line Method, this sum decreases from year to year.

Referring to Fig. 4, Sinking-Fund Method: $A\ B$ is the original investment. $A'\ B' = A\ B$ is the replacement cost. $H\ I$ is the remaining value. $I\ J$ is the accrued obligation to replace. $I'\ J'$ is the amount of the annuity allowed to the owner in consideration of his obligation to replace the worn-out article. It is the amount which he has at interest and which, together with additional annuities, will, at the end of the life term, just replace the worn-out article. $C\ D = A'\ B'$ is the amount of the replacement annuity at the end of the life of the article.

The owner is regarded as receiving back no part of his invested capital, but only a replacement annuity such that this annuity at the same rate of interest as earned by the property, of which the article forms a part, will amount to the capital to be invested in a new article at the end of its life.

In its results, this is equivalent to the Equal-Annual-Payment Method, and, owing to the simplicity of its application, should replace it.

Under the Sinking-Fund Method of estimating depreciation, it is no concern of the public what the owner does with the allowance for depreciation which he may have received. It is immaterial, so far as the public is concerned, whether this is actually set apart in a depreciation fund and is included as a part of the property which is in use, or whether, as will ordinarily be the case, the whole or a part of this fund is re-invested in extensions or betterments, or is entirely withdrawn from the business. The fact remains that this sum, real or imaginary, is an essential and integral part of the plant on which the owner is entitled to a return just the same as on the remaining value of the property.



EQUAL-ANNUAL-PAYMENT METHOD

(Amortization Computed by Sinking-Fund Methods)

FIG. 3.

The earnings of this fund are necessary additions to the same, in order that at the end of the useful life the accumulations in the fund will be equal to the replacement requirement.

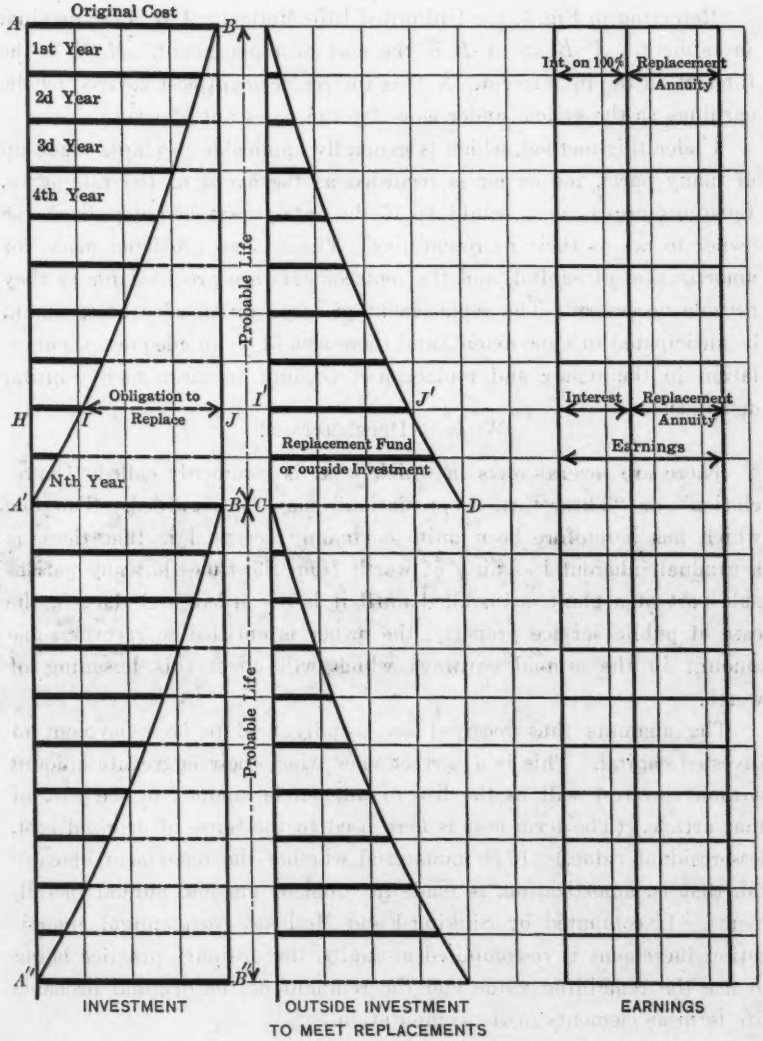
Depreciation in this sense is not "amortization" of invested capital, but is that sum which, together with the scrap value of the old appliance, will replace it with an equivalent new appliance.

There will be no capital returned to the owner, unless, of course, there is an over-estimate of the annual replacement increment, in which event the excess is available to offset early losses or to reduce invested capital.

The annual replacement increment (or what is at present commonly referred to as the depreciation allowance) should, as has been explained, ordinarily be assumed to be earning interest at the same rate as the entire business. If a water-works or a gas plant earns 7%, the same rate of interest is earned by the replacement increment, if the same be invested in large part in the business, as it should be, whether for extensions, or betterments, or operation. In view of uncertainties, however, relating to earnings, and in view of the further fact that a part of the replacement fund is sometimes actually invested in securities yielding a low interest return, there is sometimes good reason for assuming a low rate of return in estimating the annual depreciation allowance.

There should be no restriction on the owner, relating to the way in which he uses the replacement fund, except only with the limitation that enough thereof shall at all times be available to make the replacements as these become necessary.

The amount which should at any time be in the replacement fund. will, as shown in Fig. 4, be equal to the owner's accrued obligation to replace worn-out parts at the end of their lives, or, in other words, it will be equal to what is usually regarded as the accrued depreciation. The interest on the remaining value plus the interest at the same rate on the amount in the replacement fund will be the same in every year of the article's life (assuming failure to occur at the end of the probable life term), because the remaining value plus the replacement fund will at all times be 100% of the investment. The owner, therefore, as already stated, is entitled to earn a reasonable return on a 100% valuation (no deduction for depreciation) and also to earn an



SINKING-FUND METHOD

FIG. 4.

annuity which, together with interest thereon, will replace each article of which the plant is made up when it goes out of use.

Referring to Fig. 5, the Unlimited-Life Method: $A\ B$ is the original investment. $A'\ B' = A\ B$ is the cost of replacement. $M\ N$ is the interest on the investment. $N\ O$ is the replacement cost covered by the earnings as the article under consideration goes out of use.

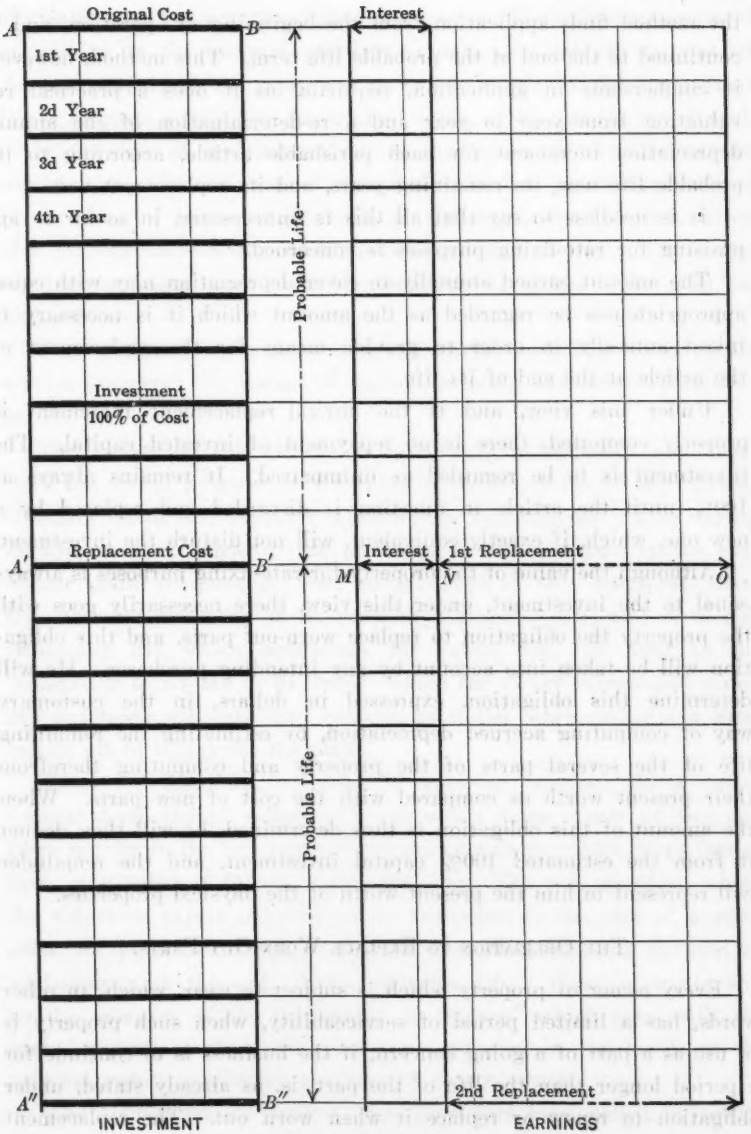
Under this method, which is generally applicable to plants made up of many parts, the owner is regarded as the agent of the rate-payer. Earnings are as they would be if the rate-payers had deputized the owner to act as their representative. There is no provision made for amortization of capital, and the replacements are provided for as they become necessary. The replacement of large units, of course, should be anticipated to some extent, and there should be an adequate accumulation in the repair and replacement account to meet such unusual demands.

What is Depreciation?

There are several ways in which what is commonly called "Depreciation" or "Theoretical Depreciation" may be regarded. The one which has heretofore been quite commonly accepted is that there is a gradual inherent lessening of worth from the time that any perishable part of a plant is installed until it is discarded, and that, in the case of public service property, the owner is entitled to recover some amount in the annual earnings which will offset this lessening of worth.

The amounts thus received are usually held to be repayment of invested capital. This is a correct view when their aggregate amount without interest will in the life of an article amount to the cost of that article. (The term cost is here used in the sense of original cost, less residual value.) It is immaterial whether the repayment of capital, that is, amortization, is made in equal or unequal annual installments. If computed by Sinking-Fund Methods, each annual amortization increment is re-computed annually, the ordinary practice being to use the remaining value and the remnant of the original probable life term as elements in the computation.

The repayment of capital in unequal annual amounts computed by sinking-fund methods is advocated by the Society's Special Committee on Valuation. This conforms to the layman's way of looking at the problem, and is sound in every particular, provided, of course, that



UNLIMITED-LIFE METHOD

(Extreme Case for a Single Article)

FIG. 5.

the method finds application from the beginning of operation, and is continued to the end of the probable life term. This method, however, is cumbersome in application, requiring as it does a practical revaluation from year to year and a re-determination of the annual depreciation increment for each perishable article, according to its probable life new, its remaining years, and its replacement cost.

It is needless to say that all this is unnecessary, in so far as appraising for rate-fixing purposes is concerned.

The amount earned annually to cover depreciation may with equal appropriateness be regarded as the amount which it is necessary to invest annually in order to provide means for the replacement of the article at the end of its life.

Under this view, and if the annual replacement increment is properly computed, there is no repayment of invested capital. The investment is to be regarded as unimpaired. It remains always at 100% until the article in question is discarded and replaced by a new one, which, if exactly equivalent, will not disturb the investment.

Although the value of the property for rate-fixing purposes is always equal to the investment, under this view, there necessarily goes with the property the obligation to replace worn-out parts, and this obligation will be taken into account by any intending purchaser. He will determine this obligation, expressed in dollars, in the customary way of computing accrued depreciation, by estimating the remaining life of the several parts of the property and computing therefrom their present worth as compared with the cost of new parts. When the amount of this obligation is thus determined, he will then deduct it from the estimated 100% capital investment, and the remainder will represent to him the present worth of the physical properties.

The Obligation to Replace Worn-Out Parts.

Every owner of property which is subject to wear, which, in other words, has a limited period of serviceability, when such property is in use as a part of a going concern, if the business is to continue for a period longer than the life of the part, is, as already stated, under obligation to renew or replace it when worn out. The replacement may be accomplished by the substitution of a new part exactly the same in all respects as the original one or the worn-out part may be replaced by apparatus of higher efficiency, of greater durability,

of greater cost, and perhaps of greater capacity. The obligation assumed in relation to the original property is only so much of the cost of the new apparatus or appliance as is equivalent to the investment in the original article, less its residual value, at the time it is replaced.

In the case of replacement due to inadequacy, this value may be considerable. In the case of destruction by fire, by violence, or as in the case of some article such as the small, wrought-iron pipes of a growing water-works, whether discarded owing to lack of capacity, or due to being worn out, the residual or scrap value may be insignificant.

The "obligation to replace worn-out parts" goes with every public service property. No owner of such a property can escape this obligation.

The growth of this obligation is entirely independent and apart from the actual progress of the inherent deterioration of any article. Ordinary repair and adequate maintenance will keep the article during its life at 100% efficiency—as measured by the standard of the particular plant in question; but, though kept in this condition, there is a constant lessening of its probable remaining life, and it is this remaining life which determines the extent and the limit of the "obligation to replace or to renew" which the owner carries and which any purchaser will take into account, as stated, when estimating what he can afford to pay for the plant.

There is, therefore, no need of knowing whether the so-called "actual depreciation" follows a curve which drops slowly at first and more rapidly in the last years of service, or a curve which drops fast at first, or whether it follows a straight line. The only question which the valuation expert is called on to determine in the case of a valuation for purchase or sale is that of remaining life and the cost of replacement at the end of the probable life. These elements will be the measure of the "obligation to replace" which goes with the plant, regardless of whether the same remains in the hands of the original owner or whether acquired by a new owner.

The "obligation to replace" has heretofore been called by nearly all writers the "accrued theoretical depreciation" or simply "accrued depreciation" and frequently has been treated as though it had actually been a repayment of invested capital.

From an accounting standpoint, there are various ways in which theoretical depreciation may be estimated and applied in amortizing invested capital. It is unnecessary to review them. Considering physical properties alone, it is always true, on the assumption of permanency of unit costs, that the "present value" plus the "accrued obligation to replace" is just equal to the investment.

The owner is at all times entitled to a return at a fair rate of interest, both on the present value and on the obligation which he has assumed to keep the plant at 100% valuation. Consequently, when rates are to be fixed, there is no need of attempting to separate the investment into its two parts "present or depreciated value" and "accrued depreciation", of which the latter might better be called "the outside investment necessary to provide for replacements." There is nothing to be written off under this method of treatment from the capital account for "accrued depreciation"; the accrued depreciation being nothing else than an estimate of the amount of an "investment necessary to provide for replacements" and this may be regarded as the "accrued obligation to replace", expressed in dollars.

THE PRESENT OR REMAINING VALUE AS AN ELEMENT IN REGULATING RATES.

When a property is to be purchased, a prudent purchaser will make inquiry relating to the condition of the various parts, in order to ascertain the amount of deferred maintenance and the probable additional term that the perishable parts of the property will serve. He will also ascertain the probable cost of replacing each part, with due allowance for residual values. He will thus obtain a measure of the obligation, which he assumes, to replace worn-out parts and to keep the plant at 100% efficiency. He will, as here indicated, treat the "obligation to replace worn-out parts" just the same as he will treat deferred maintenance. Both of these items he will deduct from "cost to reproduce new." The remainder is what he can afford to pay for the physical plant.

There is, of course, nothing wrong in using the value of physical properties as determined in this way (practically investment-less-depreciation) as a basis for the regulation of rates, provided that methods are correctly and continuously applied throughout the entire life or probable life term of each perishable article; but there will be from year to year a change in the remaining life of the several

parts of a plant, with a consequent change of remaining value and a change in the annual depreciation increment. Computation by any method requiring an annual re-determination of depreciated value, becomes involved and complex, and by its volume burdensome.

It is to be hoped that these methods, and particularly the method recommended in the Report of the Society's Special Committee on Valuation, already referred to, will not be generally accepted, and that the simpler and exactly equivalent Sinking-Fund Method of valuing always at 100% of the investment, as set forth in the writer's valuation paper,* will be substituted for the Committee method, or that, better yet, the Unlimited-Life Method will be adopted whenever conditions favor its application.

Valuation at 100% of the Investment as an Element in Regulating Rates.

The application of these simple principles, which involve the assumption that no portion of the earnings is used to retire capital, but that a suitable part of the earnings will go into a replacement fund or into an investment for replacement purposes, can perhaps best be understood on the basis of an example:

An electric generator with a 20-year life may be used for illustration. Assume, in the absence of any accepted method of appraisal, that remaining value is determined by deducting accrued depreciation estimated by sinking-fund methods.

Suppose the generator to be a part of a light and power system, and suppose, further, that it has reached the last year of useful life and will have no scrap value. A purchaser will value the generator, if he estimates interest on a 6% per annum basis, at about 8.23% of its original value, and this is all that he will pay for it. He takes upon himself, when he buys the light and power plant, an obligation to replace the generator with a new one at the end of another year. He must then renew the investment represented by this article at 100 per cent. The obligation which he voluntarily takes upon himself at the time of his purchase to replace the generator in a year is 91.77% of the cost of a new generator. At the end of the life of the generator, he will have received in his earnings the last increment of the original investment in the generator, or 8.23% of its cost, and he will meet

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 770 *et seq.*

his obligation to continue in business by acquiring a new generator and thereby renewing the full original investment of 100% in this particular appliance.

It makes no difference whether there is only one generator, or whether there are twenty in use. The principle is always the same. Moreover, in such a simple case as that of twenty generators of all possible ages, each with a useful life of exactly 20 years, the accrued obligation to replace these generators when worn out (6% per annum interest being again used for purposes of illustration) will be 41%, as shown by the writer's formula.*

In other words, the remaining value of the twenty generators (cost less depreciation, as ordinarily noted) will be about 59 per cent. No purchaser would include the twenty generators in his valuation of the property at more than 59% of their aggregate cost; but he would, nevertheless, and with reason, expect to be allowed to earn interest on 100% of their cost new, claiming rightfully that he is entitled to the same rate of income return on accumulated annual replacement increments as he is entitled to earn on the rest of his invested capital. He will justify this claim by pointing out that the earnings on any accumulated replacement fund are not available for any other use than the replacement of worn-out property; that such earnings do not, therefore, represent income; and that it is for this reason he is entitled to have the aggregate annual replacement allowances, together with accumulated interest, treated as interest-bearing capital.

In this connection attention may be called to the laws of California,† which prescribe that the Railroad Commission (having the duties of a Public Service Commission)

"may from time to time ascertain and determine and by order fix the proper and adequate rates of depreciation of the several classes of property of each public service utility. Each public service utility shall conform its depreciation accounts to the rates, so ascertained, determined and fixed, and shall set aside the moneys so provided for, out of earnings and carry the same in a depreciation fund, and expend such fund only for such purposes and under such rules and regulations, both as to the original expenditures and subsequent replacement as the Commission may prescribe. The income upon investments of moneys in such fund shall likewise be carried in such fund."

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 780.
† Statutes of 1911, 1st Extra Session, Ch. 14, Sec. 49.

It is here apparently recognized that the depreciation fund is to be used for the specific purpose of replacing worn-out parts, and that, in regulating rates, consideration need be given only to the amount of capital reasonably and properly invested, without any deduction for depreciation.

The earnings, in other words, to be adequate, must include operating expenses of every character, a proper allowance for present and prospective replacements (so-called depreciation), and a reasonable income computed at the proper interest rate on the invested capital.

It has been amply demonstrated in the writer's paper on appraisal, that this method of valuing for rate-fixing purposes at 100% of the invested capital is identical in its results with Method No. 3 of that paper, which is the method of computing depreciation recommended in the report of the Special Committee on Valuation, and which, while apparently conforming to the Court decisions, is yet like others not strictly correct unless it has been applied from the beginning. When these methods have not found application from the beginning of operation, the transition is easy from either of them to the assumption that a public service property has unlimited life and that all replacements are to be treated the same as ordinary repairs and maintenance.

According to the system of valuing properties for rate-fixing purposes at 100% of the reasonable and proper investment, it is not necessary to ascertain the accrued depreciation or the equivalent amount that should be in a replacement fund, or that should be regarded as invested to provide for renewals. It is necessary, however, in estimating earnings, to determine how much thereof should be allowed to the owner to compensate him for assuming the obligation to replace. This, if estimated for a new plant under the Sinking-Fund Method, is the annual increment determined by sinking-fund compound-interest methods, which, in the years of the probable useful life at the same rate of interest as is earned (net) by the entire property, will amount to the cost of replacing the parts of the property when worn out.

The allowance for the obligation to replace, when operating on the assumption of unlimited life of the entire property, is the actual annual replacement requirement, which need not be closely estimated, because the surplus of any one year will be available for use in subsequent years.

If the owner of a plant of numerous parts and all possible ages is required to place all sums earned as "depreciation" or earned as a "replacement annuity", in a special fund, the theoretical balance on hand in such a fund will be, as here noted:

Under the Straight-Line Method of estimating amortization for plants of full growth, the accumulation will be 50% of the investment in perishable property; this accumulation is independent of the interest rate, and the earnings by this fund are of no concern to the rate-payer.

Under the method of estimating the "accrued obligation to replace" by the Sinking-Fund Method for old plants, with interest at 6%, and the hypothesis that actual life will conform to probable life,* the theoretical accrued obligation to replace will be:

For articles having			5	-year	life,	about	38%.
"	"	"	10	"	"	"	40%.
"	"	"	20	"	"	"	38%.
"	"	"	30	"	"	"	34%.
"	"	"	40	"	"	"	31%.
"	"	"	50	"	"	"	28%
"	"	"	60	"	"	"	25%.

These percentages apply strictly, of course, only to cases in which the age of the plant is greater than the life of its parts. They indicate the extent, however, to which an accumulating replacement or depreciation fund may exceed the demands on it. In some measure, approximating this unexpended surplus, such a fund is available for outside investments. Interest on the same, when it represents the obligation to replace, must be earned by the owner for the benefit of the plant; consequently, it makes no difference to what purpose he applies this surplus, provided only that ample assurance remains that replacements will actually be made when necessary.

When the capital is being amortized, the accumulations in the amortization fund are the absolute property of the owner. They are not subject to control by the rate-payer, yet, nevertheless, here, too, proper assurance may be expected that replacements will be duly made and it would not be unreasonable to expect an adequate fund to be maintained to meet emergencies.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, formula on p. 784.

In order to show beyond all question that the method of valuing at 100% in fixing rates under continuous application from the beginning is correct and proper, an electric generator with a life of 20 years, which has served 15 years, may be taken as a basis for an illustration. The usual assumption is made, for the purpose of this illustration, that there is no change in the cost of this article during its life and that it will go out of service when exactly 20 years old. Interest in this illustration is taken at 6% per annum:

Equal-Annual-Payment Method.—(Always correct if applied from the day the article went into use.)

Original investment		$100.00
Life (new)	20 years	
Time in service	15 years	
Remaining life	5 years	
Accrued depreciation (= amortization to date)		63.27
Remaining value		36.73
Interest on remaining value	$2.20	
Annual depreciation or annual amortization increment for 16th year	6.52	
Required net earnings	$8.72	

In this case the depreciation in the 16th year is that amount which invested annually at 6% will retire the remaining value $36.73 in the remaining 5 years of life.

Sinking-Fund Method.—(Method No. 1, as explained in the writer's paper on Valuation.*)

Permanent investment		$100.00
Life (new)	20 years	
Time in service	15 years	
Remaining life	5 years	
Interest on the investment	$6.00	
Annual depreciation or annual replacement increment for any year	2.72	
Required net earnings	$8.72	

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

Although it can thus be shown that for each year the earnings should be $8.72 on the $100 of original capital investment, a new calculation is necessary for each perishable article and for each year under the Equal-Annual-Payment Method, and for the rational Sinking-Fund Method a single simple calculation suffices for each group of articles of the same probable life term.

Effect of Inadequate Earnings.

Depreciation or replacement annuities are not ordinarily fully covered by the earnings in the early years of the life of public service properties. The cases in which earnings are adequate from the beginning to yield a proper interest return, together with the replacement annuity, are relatively few. Consequently, particularly if methods of appraisal other than the Unlimited-Life Method are insisted upon, there may be a period in the life of the public service concern during which a part of the so-called accrued depreciation will remain unrecovered. Temporarily at least, there will then appear to be a business loss; but a loss of the character which, in the course of time, should be amortized if the business enterprise is a legitimate one.

The United States Supreme Court, in Knoxville *vs.* Knoxville Water Company, says:

> "If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of the unwarranted dividends upon over-issues of securities, or of omission to exact proper prices for the output, the fault is its own."

In expressing this opinion, the Court has overlooked the fact that such earnings as it assumes to have been possible can rarely be realized at the beginning of a new business, and that there will be some years in the life of almost every public service concern for which it would be unjust to make so sweeping an assumption.

Although it would be improper to consider early losses, including deficient or unrecovered depreciation, in every case, as the exact amount to be amortized by future earnings, there is no doubt that due consideration should be given to such losses in order that, from the sum of experience, the world over, a proper basis may be established for making the necessary allowance therefor. This allowance, of course, can be in the nature of an interest-bearing addition to the invested capital.

It does not, strictly, represent value until a return thereon in the earnings is assured.

It is to be expected, therefore, that in the case of many public service properties the owner will have had no opportunity to make investments the earnings of which will meet his replacement requirements. When there has been such a deficiency of earnings, the ordinary method of computing the annual replacement increment by sinking-fund compound-interest methods, based on the original probable life or on the reduced value computed by any method and the remaining life, would be improper and inadequate. It is under such circumstances that recourse must be had to special methods of computing the depreciation or the annual replacement increment.

Unlimited-Life Method.

In such cases as those just referred to it may be necessary to provide for a replacement at the full cost of various parts in the remaining years of life, or of burdening the property with an allowance for the accrued deficiency of earnings, which is then to be amortized in a reasonable number of years.

When an appraisal is to be made, for rate-fixing purposes, of a plant which has continuously been operated with inadequate earnings, there is no escape from the conclusion that the investment has remained undiminished. It would be unreasonable not to concede to the owner his right to a return on the investment and also to allow him to earn enough in addition to meet replacements as they become necessary. Considerations of this character led the writer to adopt the Unlimited-Life Method as not only fair to both owner and rate-payer, but as the one method which has the peculiar advantage of requiring lowest earnings in the early years of operation when the rate-payers are fewest in number.

This method has the advantage over all the other methods that it is applicable regardless of past conditions. All the other methods, the Straight-Line Method of estimating depreciation, the Equal-Annual-Payment Method, the Sinking-Fund Method, are only then correct and equitable methods of making appraisals and of determining annual depreciation, or amortization increments, when they are properly applied from the beginning of operation. Their use assumes continuous appli-

cation throughout the entire probable or actual life of each part of a plant.

When the Court says that, if an owner fails "to exact sufficient returns to keep the investment unimpaired, whether this is the result of the unwarranted dividends * * * or of omission to exact proper prices for the output, the fault is his own", this fact is overlooked. It is time that the Courts revise such decisions, as they have already caused much trouble and no little juggling on the part of experts to secure justice for their clients.

There can be no fairer nor better way of determining the earnings of a public service property than that under which the owner is regarded as the agent of the public. He is then to be allowed interest on the investment which he makes as such agent, and he is also to be allowed whatever may be necessary for the up-keep of the property, maintenance, repairs, and replacements.

There should be no amortization of capital, no more than there would be if the rate-payers themselves made the investment without the intervention of an agent.

If this view is adopted, every public service property may be considered to have unlimited life. The property in its entirety is not subject to depreciation. Its appraisal at all times for rate-fixing purposes is at the full amount of the investment.

The earnings must be adequate to provide interest on the full amount of capital invested and also to meet the replacement requirements. The annual replacement requirement will be negligible in the early years of a plant's life, or in the early years of any single article; but this replacement requirement will grow as the plant acquires age. It can be tentatively estimated. All sums collected for replacements should be separately accounted for. Ordinarily, the replacement of larger units will be anticipated, and their cost will be distributed over a number of years, and, in the long run, particularly when a complex plant worth millions is under consideration, there should be no difficulty in estimating a fair average annual replacement requirement.

What Part of Earnings May Be Written Off Capital?

Under the impression that any sum collected under the designation "depreciation" or "amortization" is a repayment of a part of the investment, such sums are frequently written off from the in-

vested capital or from the "cost of reproduction new", which is an approximation of the invested capital.

This is correct:

1st.—When, from the beginning of the installation, the Straight-Line Method of estimating "amortization" or "depreciation" is applied and the amortization allowance has actually been earned.

2d.—When the actual amount of the annual decrease in remaining value has been computed by the Equal-Annual-Payment Method and earned from the beginning.

3d.—When any other definite scale of capital repayment is adopted such that in the life of the part the several payments "without interest thereon" amount to 100 per cent.

Value from Various Standpoints.

Under the Straight-Line Method of computing depreciation or of amortizing capital, the value for rate-fixing purposes, "cost-less-depreciation", is the same as value to a purchaser. This is self-evident.

Under the Equal-Annual-Payment Method, the appraisal for rate-fixing purposes is the cost, less accrued depreciation; and here, too, this value is the same as that which would be determined by a purchaser.

Under the Sinking-Fund Method, the appraisal for rate-fixing purposes is at 100% of the investment. It is assumed, under this method, that the replacement fund is intact and a part of the property. Thus considered, the property (including the replacement fund) has a value to a purchaser of 100 per cent. If he does not pay 100% and does not get possession of the replacement fund, then he must put it up himself, because he takes with the property, as has been explained, the obligation to replace, which, expressed in dollars, is the amount which should be in the replacement fund.

Under the Unlimited-Life Method there has been no return of capital. The appraisal is at 100% of the investment. The value of the property to a purchaser is 100%, because he takes the place of the original owner and is entitled to receive interest on the full amount of the investment. The earnings yield him interest on this amount, and he needs no replacement fund, because the replacements will be met out of earnings from year to year as they have to be made.

It thus appears that value for rate-fixing purposes and value to a purchaser can be reconciled, regardless of the method adopted for computing adequate earnings.

APPRAISAL MUST BE MADE WITH DUE REGARD TO PAST HISTORY OF THE PROPERTY AND TO PROBABLE FUTURE CONDITIONS.

A change from one method of computing allowable earnings to another is not always possible without injustice to either the rate-payer or the owner. If the Equal-Annual-Payment Method has been acceptable for a time and it is then determined to pass to the Straight-Line Method, there will be an immediate reduction of the interest-bearing portion of the investment. The cost-less-depreciation will be smaller under the latter than under the former, and the owner will be compelled to sacrifice a part of his investment without compensation.

If the Sinking-Fund Method has been used in computing allowable earnings and there is a change made to the Straight-Line Method, this will release the accumulation in the replacement fund, which is thereupon to be regarded as returned capital, but the replacement fund, whether it exists as a fund or has been otherwise used by the owner, will fall short of the accrued depreciation, as the same would have been estimated by the Straight-Line Method, and, consequently, the owner will again be unfairly deprived of a part of his investment.

If the Sinking-Fund Method has been used from the beginning of operation and a change is made to the Unlimited-Life Method, there may be an accumulation in the replacement fund, or there has been distributed to the owner from this fund a larger sum than is necessary to aid in equalizing the annual actual replacement requirements, and to this extent the owner would benefit by the change.

If operations have been conducted from the beginning on the basis of earning a proper interest return on the investment, plus the amount necessary from year to year to renew worn-out parts, and there is then to be an appraisal and computation of rates by any method other than the Unlimited-Life Method, this would be an injustice to the owner, because he has not in the past had the benefit of the larger earnings which he should have had under these other methods of computation.

All this is referred to, to show that the correctness of any method of appraisal for rate-fixing purposes is not predicated on the method alone, but on the continuous application of the adopted method from the beginning, and on the assurance that the same method will be continued in the future. The decisions of the Courts cannot change

this fact, and, where they have disregarded it, they will undoubtedly be modified in the course of time.

Comparison of Actual with Probable Life.

Attention has been called* to the fact that the expectancy of articles which remain in use, instead of probable life new, should be taken into account whenever an article has acquired some age and has survived the possible accidents of the early period of its life. In other words, it is proper to pay attention to the condition of the plant and to the condition of the articles of which it is made up when remaining value is to be estimated.

All estimates of annual depreciation and of accrued depreciation are based on premises which cannot be determined with accuracy. The probable life of any article when new and the life expectancy of any article which has been in use for some time cannot be determined with any great degree of precision. Consequently, all estimates of depreciation are only approximations.

There is so much uncertainty in such estimates, under the ordinary conditions under which public utilities are operated, that a wide range in the method of making the estimates has been the result.

When, therefore, the correctness of methods is under discussion, this fact should not be lost sight of. Nevertheless, the academic discussion which is being indulged in by the Engineering Profession is justified, because it will lead to an ultimate standardization of methods and finally to the general adoption of the most convenient and generally best method of making the valuation which is to serve as the basis for fixing rates fair to both the rate-payer and the owner.

Recognizing this limitation, and all the uncertainties that result from imperfect knowledge relating to the actual and to the probable life of the elements of any public service plant, and to the difficulty of determining the expectancy of those elements which have already been in use for some time, it is nevertheless important that the whole question be fully considered from the theoretic standpoint, in order that a framework may be constructed into which the best data furnished by experience can be fitted.

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* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 837-838.

The non-agreement of the actual life of individual items with the probable life and the extent to which this should be taken into account in estimating present worth and in estimating replacement requirements have, in line with this thought, been studied on various assumptions with interesting results. These will be briefly referred to, the tables that were prepared being withheld except as to final results until more definite information is available relating to the probable rate of failures from year to year with advancing age in any group of articles.

When any large number of articles which have the same probable life, as, for example, 10 years, are considered, there will be as many aggregate service years represented by the failures to reach the probable life term of 10 years as there will be service years represented by those articles which outlast the 10-year term. It may also be accepted as a certainty that there will be a greater number of articles per year to go out of use in the years just preceding and just following the term limit than at any other time.

This is in conformity with the law of probabilities.

On this hypothesis, with the assumption of failures according to the law of probabilities, but all within a period equal to twice the probable life, and on another hypothesis according to which a gradual uniform increase of failures up to the tenth year and thereafter a gradual uniform decrease of failures to the twentieth year—or to twice the probable life term, it was found possible to predict expectancy and also to compute the annual replacement requirement.

Unfortunately, there are no records available to justify the assumption that all failures may be assumed to occur within twice the probable life term. Nevertheless, the distribution of failures to the double term, with a greatest rate of failures at the end of the probable term, is certain to be far nearer the truth than the usual assumption of general agreement between probable and actual life. It may be proper, therefore, to refer to the results of the studies.

Under the assumption of failures according to the law of probabilities, it was found that an article having a probable life of 10 years when new should have the expectancies shown in Table 1.

On the assumption that the rate of increase in the annual number of failures is uniform from the beginning of service to the end of the probable life term, and that there is a similar uniform decrease

in the annual number of failures after the end of the probable life term, Table 2 has been prepared, and shows the expectancy for any surviving article that is still in good condition.

TABLE 1.—EXPECTANCY, ACCORDING TO THE LAW OF PROBABILITY, FOR AN ARTICLE WITH A 10-YEAR PROBABLE LIFE; ON THE ASSUMPTION THAT ALL ARTICLES FAIL WITHIN A PERIOD PRACTICALLY TWICE THE PROBABLE LIFE TERM.

No. of years.	Expectancy, in years.	No. of years.	Expectancy, in years.
1	10.00	11	2.71
2	9.00	12	2.42
3	8.05	13	2.18
4	7.11	14	1.97
5	6.22	15	1.80
6	5.40	16	1.64
7	4.78	17	1.50
8	4.04	18	1.25
9	3.52	19	1.00
10	3.08		

Although, under this hypothesis of failures, there may be considerable departure from the actual number of failures in any year, there can be no question that this hypothesis is a much nearer approach to the truth than the assumption that remaining value and accrued depreciation can be estimated from tables based on probable life without regard to the condition of an article under consideration.

TABLE 2.—EXPECTANCY ESTIMATED ON THE ASSUMPTION THAT, OF A LARGE NUMBER OF ARTICLES HAVING A 10-YEAR PROBABLE LIFE, THE MAXIMUM RATE OF FAILURES IS 10% IN THE TENTH YEAR, WITH A GRADUAL INCREASE TO THIS AMOUNT FROM THE FIRST YEAR AND A GRADUAL DECREASE FROM THIS AMOUNT TO THE TWENTIETH YEAR.

No. of year.	Expectancy, in years.	No. of year.	Expectancy, in years.
1	10.00	11	3.67
2	9.09	12	3.33
3	8.27	13	3.00
4	7.46	14	2.67
5	6.77	15	2.33
6	6.12	16	2.00
7	5.51	17	1.67
8	4.95	18	1.33
9	4.44	19	1.00
10	4.00		

The basis for the results in Table 2 is shown diagrammatically in Fig. 6. The expectancy is found by dividing the remaining service years at any time by the corresponding number of surviving units. The reverse curve marked "Remaining Units" clearly indicates the hypothesis of failures on which the table is based. It is to be noted that, under this hypothesis, there is no serious departure from the results that were obtained by assuming that the law of probabilities would apply.

The replacement requirements, if failures occur substantially as assumed, would increase, for $100 of original investment, from $1 in the first year to about $10 in the ninth year, fluctuating thereafter between $9 and nearly $12 per year, and gradually settling down to $10 per year. For an annual investment of $100 per year (that is, for a growing plant) the replacement requirements would gradually increase from $1 per year in the first year to $463 in the fiftieth year, or from $1 per $100 of investment in the first year to $6.01 in the tenth year; to $8.16 in the twentieth, and to $9.27 in the fiftieth year.

In practical application, in other words, the annual replacement requirement in the case of a plant of full growth, all parts of which have a probable life of n years after the plant is n years older than any of these parts, will be about one-nth of their replacement cost.

In a plant which continues to grow, the annual replacement requirement will gradually approach, but can never quite reach, one-nth of the total replacement cost.

Although it may be granted that in the long run the failures of individual articles in any class will follow some definite law (perhaps a law similar to that which has been cited as more probable than failure always at the end of the probable life term), the fact remains that in no particular case, no matter how large a plant may be, will there be absolute conformity with any such law. Consequently, figures determined on the basis of any hypothesis of failures can be used to prepare smoothed-out curves, and from such curves tables for general use can be prepared. It is enough to know for the present that, ordinarily, it may be assumed that the replacement requirement of a large number of articles with a probable life of n years should increase progressively year by year to about one-nth of their cost in the nth year.

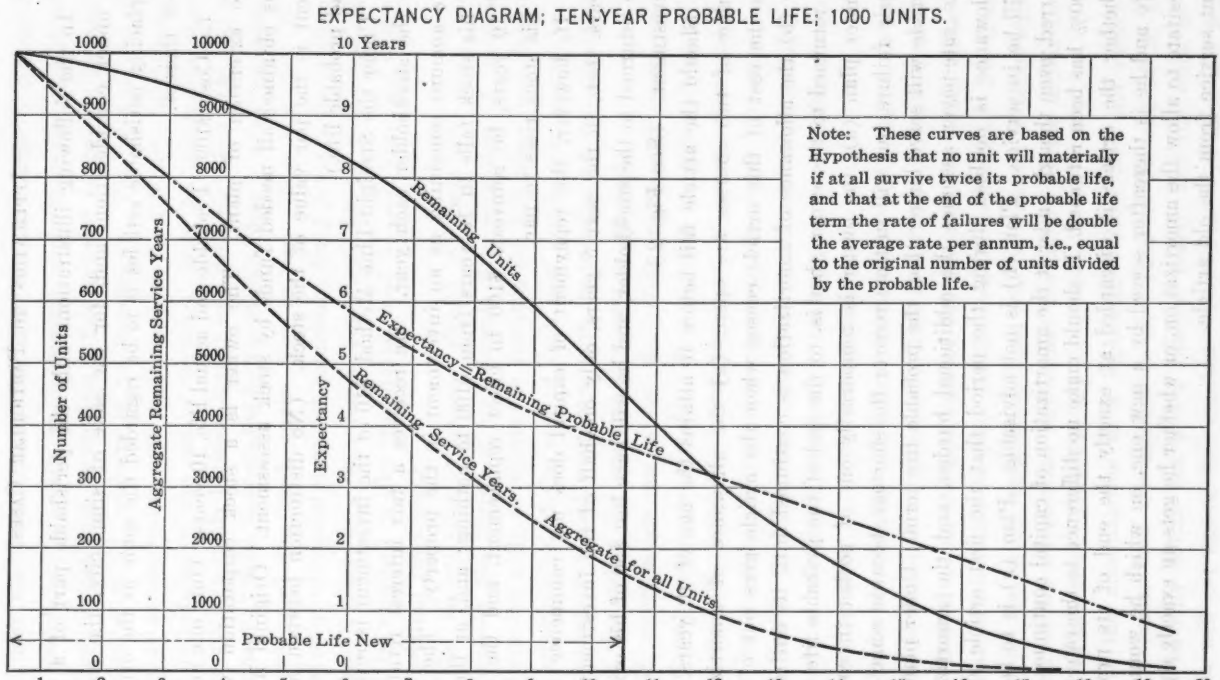
EXPECTANCY DIAGRAM; TEN-YEAR PROBABLE LIFE; 1000 UNITS.
Number of Units
Aggregate Remaining Service Years
Expectancy
10 Years
Remaining Units
Expectancy = Remaining Probable Life
Remaining Service Years, Aggregate for all Units
Probable Life New
Years
Note: These curves are based on the Hypothesis that no unit will materially if at all survive twice its probable life, and that at the end of the probable life term the rate of failures will be double the average rate per annum, i.e., equal to the original number of units divided by the probable life.

FIG. 6.

Application to Particular Cases.

In the following illustrations only the perishable part of a plant is under consideration, and, for the sake of simplicity, the cost of replacing perishable articles is to be regarded the same as the original cost thereof.

1.—Cost, $100. Probable and actual life, 10 years. Only one article. No extension of plant. The owner is a stock corporation which has obtained all needed funds by stock assessment. Original investment is the par value of the stock. (No distinction between actual and probable life.)

Under the Straight-Line Method, $10 of the investment is returned to the stockholder each year. He receives a fair interest return on the amount remaining as an investment in the property. The value of his stock falls in 10 years from $100 to nothing; and, at the end of 10 years, he reinvests $100 to effect replacement, and the value of his stock rises to par.

If, however, the repayment of capital does not commence for 5 years, then, in the next 5 years, only one-half of the investment will be returned to the stockholder, and he will have lost $50 of his original investment. (See Fig. 2.)

Should the article fail before it attains the age of 10 years, then either of two courses are open: (a) the allowance of amortization on the cost of this article ceases when the article goes out of use; or (b) the allowance of amortization is continued until its entire cost is returned to the owner, that is, to the end of the probable life term. Under plan (a), the owner is temporarily out of pocket in case of early failure, but will in time recover the shortage, because some other article will serve longer than the probable life term. Under plan (b), the rate-payer assumes the additional burden, and the amortization allowance is doubled during the period that the first article should still be in service. Plan (b) is not advisable. Plan (a) is to be preferred, even though under it the amortization of capital continues after 100% has been returned. It should make no difference to the rate-payer whether the article is discarded at exactly the end of its probable life and he is thereafter served by a new one, on which he would not hesitate to allow the amortization, or whether he gets an exactly equivalent service from the old article.

Under the Equal-Annual-Payment Method, there is returned to the stockholder annually an increasing amount of the capital which he has invested and he receives a fair interest return on the amount remaining in the property as an investment. The value of his stock falls at an increasing rate as the end of the life term is approached, and is nothing at the end of 10 years. (On the assumption that no distinction is made between actual and probable life.) On the reinvestment of $100, contributed by the stockholders to effect replacement, the value of the stock again rises to par. (See Fig. 3.) If the repayment of capital does not commence for 5 years, then in the next or last 5 years on a 6% interest basis only $57.23 of the investment will be returned to the stockholder, and he will have lost $42.77 of his original investment.

The amortization allowance should continue for every article throughout the period of its probable life, regardless of whether it is discarded early or whether it survives the full term; and then, too, the amortization of the cost of each new article that replaces a discarded article should commence with its installation and should cease when the end of the probable life term is reached.

If amortization ceases for the article which fails early and is continued during the probable life term only for those which outlast their probable age—this being the limit of amortization tables as now in use—then the method is unjust to the owner. In other words, this method is difficult and uncertain in its application. Some improvement will result, however, in taking into account the expectancy at varying ages, as elsewhere explained.

Under the Sinking-Fund Method, the owner will be given an opportunity to establish a replacement fund, and if it be actually established and treated as an integral part of the property, the stock value will remain continuously at par. At the end of the probable life term of 10 years the entire replacement fund will be required to effect replacement, on the supposition again that the failure of the article occurs just at this time. (See Fig. 4.)

If the accumulation in a replacement fund does not commence for 5 years, then the amount in this fund at the end of 10 years will be only $57.23 (on a 6% interest basis), and the value of the stock will not exceed this amount. The owner will have lost $42.77.

It need hardly be stated that here, too, expectancy should be taken into account, when a determination of remaining value is to be made.

Under the Unlimited-Life Method, the replacement will be assured at the end of 10 years and the value of the stock at all times will be at par. (See Fig. 5.)

2.—Cost, $100. Probable and actual life, 10 years. Only one article. No additions are made to the plant. The owner is a stock corporation which has constructed the plant with money obtained by a bond issue.

Under the Straight-Line and Equal-Annual-Payment Methods, bonds are retired as amortization allowance is earned. The stock has only such value as results from an interest allowance in excess of the interest on the bonds, and no value if these interest rates are the same, and if, as assumed in this illustration, there is no surplus of earnings above interest on remaining investment, plus the amortization allowance. In case that the amortization is not earned for a term of years, the ownership of stock may become a liability instead of an asset.

Under the Sinking-Fund Method, the fund which is accumulating for replacements may be used for the retirement of bonds or otherwise. The stock will again have only such value as results from interest earned in excess of interest on bonds. In the case of earnings for a term of years, which are inadequate to meet the replacement annuity, the stock may be a liability instead of an asset.

Under the Unlimited-Life Method, bonds are refunded as they become due. The amount of outstanding bonds remains unchanged. The stock has only such value as results from the excess of interest earned when compared with the interest on the bonds.

It is obvious from these illustrations that, when works are constructed under bond issues, the earnings should exceed the interest on the bonds, in order to insure some compensation for management and the hazards of the business.

3.—Cost, $100. A single article with a probable life of 10 years. Interest, 6 per cent. With due regard to condition or to the departure of actual life from probable life.

Under the Straight-Line Method, the invested capital is reduced $10 each year. At the end of the 10-year term the article is found to be still in good condition; its expectancy is then probably from 3 to 4

years. The owner has already received back his full investment. There is no remaining investment, although the remaining value of this unit is from $30 to $40. By some hard and fast rule, the $10 per year payment should continue until the article is discarded, or by some other equally inflexible rule, as under illustrations (1) and (2), the $10 per year should continue until ten payments have been made, and should cease to be paid after the tenth payment. (See Table 3.)

TABLE 3.—REMAINING VALUE AND REQUIRED EARNINGS.

Straight-Line and Equal-Annual-Payment Methods. Cost, $100. Single Article. Probable Life, 10 Years. Interest, 6 per cent. Net Earnings, 6 per cent. Applicable to a Surviving Unit.

(Expectancy on the Hypothesis Explained in the Text.)

Year.	Expectancy, in years.	Remaining Value.*		Required Earnings.	
		Straight-line method.	Equal-annual-payment method.	Straight-line method.	Equal-annual-payment method.
1	10.00	100.00	100.00	16.00	13.59
2	9.09	90.90	93.09	15.46	13.59
3	8.27	82.70	86.54	14.98	13.59
4	7.46	74.60	79.77	14.50	13.59
5	6.77	67.70	73.77	14.08	13.59
6	6.12	61.20	67.89	13.66	13.59
7	5.51	55.10	62.11	13.30	13.59
8	4.95	49.50	56.73	12.94	13.59
9	4.44	44.40	51.55	12.64	13.59
10	4.00	40.00	47.08	12.40	13.59
11	3.67	36.70	43.50	12.22	13.59
12	3.33	33.30	39.91	11.98	13.59
13	3.00	30.00	36.32	11.80	13.59
14	2.67	26.70	32.51	11.62	13.59
15	2.33	23.30	28.75	11.38	13.59
16	2.00	20.00	24.91	11.20	13.59
17	1.67	16.70	20.87	11.20	13.59
18	1.33	13.30	16.85	10.78	13.59
19	1.00	10.00	12.82	10.60	13.59
20					

* At the beginning of the tenth year, the expectancy is 4 years, consequently, the remaining value of the article under the Straight-Line Method at $10 per year is $40.

In a 10-year life sinking fund table at 6% the remaining value for 4 years of remaining life is $47.08. The annuity, which will amount to $47.08 in 4 years, is $10.77. Interest on $47.08 at 6% is $2.82. The required net earnings, therefore, are $10.77 + 2.82 = $13.59.

Under the Equal-Annual-Payment Method, the remaining value at any time may be estimated from 10-year life tables, using the expectancy as an element in the calculation. If the probable life of the article new is used throughout, this should be done only under an inflexible rule, and all amortization should cease when the article

has reached the end of the probable life term, as under illustrations (1) and (2). If, however, the condition of the article is taken into account and its expectancy is determined, the remaining value will be about as shown in Table 3.

Under the Sinking-Fund Method, if computations are made with due regard to the expectancy of the individual articles, the required annual earnings will be uniform in amount and the same as under the Equal-Annual-Payment Method.

Under the Unlimited-Life Method, the investment again remains unimpaired, and whenever the article fails, whether early or late, it is replaced with a new one, the replacement allowance to be earned being the amount already noted, in order that funds may be available when the failure occurs.

4.—Cost, $100. Numerous articles. Probable life 10 years, some articles failing early, others serving after the end of the probable life term. The plant has full growth, and there are no further additions or extensions. All computations on a 6% interest basis.

Under the Straight-Line Method, the annual allowance for amortization will be $10. At the end of the 10 years, there will be some of the original articles still in service, possibly 45 to 50 out of each 100. The others, making up the original group, having an aggregate cost of $100, have failed and were replaced with new articles and some of these have failed and were replaced. The owner has received amortization in the sum of $100, and he has reinvested in replacements somewhat more than $60. The remaining investment, therefore, is about or a little more than $60. After the first 10 years have passed, the annual replacement requirement will be just about equal to the amortization allowance, and the remaining investment will continue at some amount somewhat in excess of $60. The required earnings on a 6% basis begin with $16 per year, gradually dropping to between $13 and $14.

Under the Equal-Annual-Payment Method, the appraiser will find himself in considerable difficulty, until the time is reached when the number of annual failures is fairly uniform. The extent of the amortization will then be about $30 to $40, and the remaining investment will be about $60 to $70. The replacement requirement in this case, also, will be just about equal to the annual amortization allowance, and the remaining investment will continue at about $60

to $70. (The required earnings will be uniformly and continuously $13.59 per year.)

Under the Sinking-Fund Method, with a replacement fund made up of equal annual replacement allowances computed by the Sinking-Fund Compound-Interest Method, there will be a gradual growth of this fund in 10 years, until the number of annual failures is fairly uniform, when there will be about $30 to $40 in the fund. Thereafter, the interest on this fund, together with the annual replacement allowance, will be about $10, which is the amount necessary to meet the annual replacement requirement. The required earnings are, from the beginning and continuously, $13.59 per year. The replacement fund will gradually grow until it contains about $30 to $40. This fund will be available for outside investments.

Under the Unlimited-Life Method, the $100 of original investment would remain unimpaired, and there would have to be provided a growing annual amount for actual replacements, beginning with about $1.00 the first year; $2.01, the second; $3.04, the third; and so on, to about $10 in the tenth year and thereafter.

5.—The case remains to be considered of a plant which has not attained full growth, and to which additions are made from year to year.

From a tabular comparison it appears that, when computed with regard to the expectancy of surviving articles, the application of the Unlimited-Life Method has the particular advantage of requiring lowest earnings in the early years, and the Straight-Line Method has the disadvantage of requiring the highest earnings in the early years; the Equal-Annual-Payment Method holds an intermediate position. The required earnings, computed on the hypothesis of failures as heretofore stated, would be $7.00, $16.00, and $13.59, respectively, in the first year; $120.12, $143.38, and $135.90, respectively, in the tenth year; and $283.18, $281.36, and $271.81 respectively, in the twentieth year.

Comparison of Methods.

When a municipality constructs improvements under a bond issue, suitable provision is made for the replacement of any of the worn-out parts of these improvements at the time that such parts go out of use. This is in strict conformity with the proceedings under the Unlimited-Life Method; but, in the case of the municipality, the

bond issue is also to be taken care of. The cost of the improvements is, in other words, to be distributed fairly to those who will benefit by them. This is usually done by fixing the term of the bonds so that the cost will be distributed over a sufficiently long period of time. The determination of this time period need not be in any definite relation to the life of the parts of the improvement. The improvement itself will usually be one that may be regarded as having unlimited life, such as parks, playgrounds, streets, and the like. When the term of the bonds of longest life has been fixed, on the basis of the probable life of the main elements of the improvements or otherwise, then a determination must be reached as to the best and most equitable rate of amortization.

This amortization may take place at a uniform rate per year, bearing heavily on present-day property owners—Straight-Line Method.

It may take place at an increasing rate per year:

(a) According to the scheme outlined under the Equal-Annual-Payment Method; or

(b) According to any arbitrary scheme that will approximate the compound-interest Sinking-Fund Method of estimating the annual amortization increment.

Or, it may be deferred for a time and then take place according to either of these methods.

In the case of a public service property constructed by a municipality, the amortization of capital usually begins at or soon after the acquisition of the property; and in the case of a utility constructed by a private owner, the amortization of capital should begin under the Straight-Line and Equal-Annual-Payment Methods at the beginning of operation, and, under the Sinking-Fund or the Unlimited-Life Methods, as herein fully explained, not at all during the continuance of private ownership.

In order to make clear how unsatisfactory it is to have to estimate "depreciation", or, strictly speaking, "the replacement requirement", according to any method except that of unlimited life, the diagrams on Fig. 7 have been prepared.

The curves here shown afford a comparison of replacement requirements determined according to some reasonable hypothesis relating to the expectancy of surviving articles, with the amortization allowance

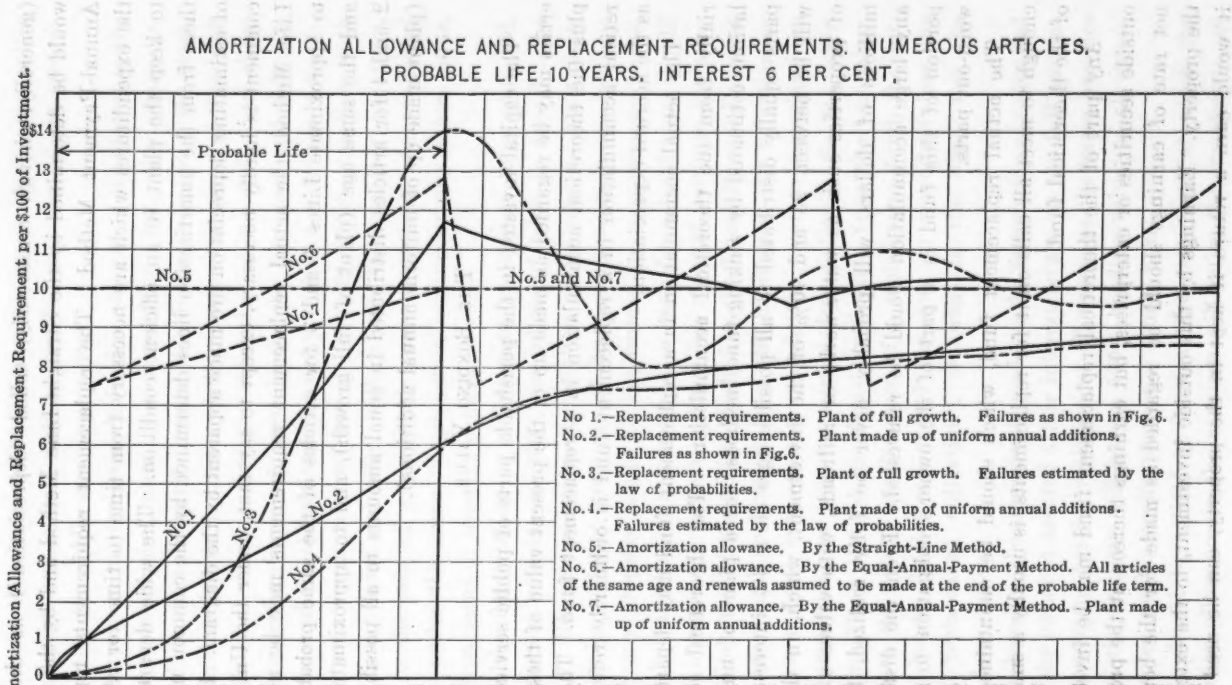
AMORTIZATION ALLOWANCE AND REPLACEMENT REQUIREMENTS. NUMEROUS ARTICLES.
PROBABLE LIFE 10 YEARS. INTEREST 6 PER CENT.
Amortization Allowance and Replacement Requirement per $100 of Investment.
Probable Life
No.5
No.6
No.7
No.5 and No.7
No.1
No.3
No.2
No.4
No 1.—Replacement requirements. Plant of full growth. Failures as shown in Fig. 6.
No. 2.—Replacement requirements. Plant made up of uniform annual additions. Failures as shown in Fig. 6.
No. 3.—Replacement requirements. Plant of full growth. Failures estimated by the law of probabilities.
No. 4.—Replacement requirements. Plant made up of uniform annual additions. Failures estimated by the law of probabilities.
No. 5.—Amortization allowance. By the Straight-Line Method.
No. 6.—Amortization allowance. By the Equal-Annual-Payment Method. All articles of the same age and renewals assumed to be made at the end of the probable life term.
No. 7.—Amortization allowance. By the Equal-Annual-Payment Method. Plant made up of uniform annual additions.
Years

FIG. 7.

(generally, though erroneously, called "depreciation") as the same would be determined by the Straight-Line Method and by the Equal-Annual-Payment Method. The replacement requirements represent the expenditures which are necessary from time to time for renewals to keep the plant in an efficient condition. The wide departure of these from the amortization lines determined by the common methods of estimating depreciation are made apparent by the diagram. Further comment is hardly necessary, except to say that, under the Unlimited-Life Method, the actual replacement requirements may be assumed to approximate Lines 1 and 3 for articles of the same probable life and the same age (plant of full growth), and to approximate Lines 2 and 4 for articles distributed in equal amounts to all possible ages (plant made up of uniform annual additions).

The Bonding Value.

The capital invested in the perishable parts of public service properties may be regarded as made up of the present value of these parts plus the theoretical accumulation in a replacement fund. The theoretical accumulation in a replacement fund is ordinarily referred to as the accrued depreciation.

The actual accumulation in a replacement fund may depart materially from the theoretical accumulation. The theoretical accumulation, estimated by sinking-fund methods, for a plant of numerous parts equally distributed to all possible ages on a 6% interest basis, will be between 25 and 40% of the investment, which, in the case of properties such as water-works or railroads, with values of many millions of dollars, will probably never be fully realized, because any large accumulation would be reinvested. Even the distributed portion of this fund is a part of the owner's obligation to replace worn-out parts.

The actual replacement fund, which should be maintained large enough to meet all calls on it for replacements, is usually a small part of the theoretical fund.

Any part of the theoretical replacement fund may be invested in outside securities or enterprises, but earnings thereon at the property's net rate of earnings should be regarded as made for the benefit of the property. Earnings by this outside investment, to the extent that it would earn a return if kept in the property, can be used for no

other purpose than to make replacements; consequently, the entire theoretical replacement fund may be regarded as a part of the owner's investment in the public service property.

The surplus in the replacement fund is sometimes distributed to the stockholders when the owner is a corporation; but, when thus distributed, it is not a dividend, because it reduces the value of the stock. When the distribution is made, whether under the name of dividend, or not, the fact remains that the obligation of the individual share of stock to replace worn-out parts is thereby increased and that any such increase of the obligation must carry with it a lessening of worth.

It is not correct to consider an annuity which, together with interest compounded annually, will, within the probable life term, amount to the cost of a perishable part of a plant, when collected annually under the name "depreciation" or under any other name, to be a repayment of invested capital.

When the depreciation or replacement annuity is thus computed, the investment remains at 100% regardless of what is done by the owner with the accumulating depreciation or replacement allowances.

A bond issue should never exceed the present worth, as the present worth of any property would be determined by a purchaser; consequently, the valuation of a property as a basis for a bond issue should take into account the method under which required earnings are computed. When the Straight-Line Method or the Equal-Annual-Payment Method has found application from the beginning, the valuation as a basis for a bond issue should include the "cost to reproduce the physical properties new less the accrued depreciation". So, too, when the Sinking-Fund Method has found application from the beginning, but without the actual establishment of a replacement fund, the outstanding bonds should not exceed the cost to reproduce less the "accrued replacement obligation", or, what is the same, less the accrued depreciation.

When, however, the appraisal is made on the assumption of unlimited life and the owner is assured a fair interest return on the full amount of his investment besides replacement allowance as the same becomes necessary, the property will have a 100% bonding value, as all extensive, complex, public service properties should have.

Bonds should be retired at least at the same rate at which the amortization allowance in the earnings will retire the investment, or,

in the case of the Sinking-Fund Method, at the same rate at which the obligation to replace grows.

In a property all parts of which have a limited life, when earnings are estimated on any other assumption than that of unlimited life for the entire property, the outstanding bonds may approach the 100% limit when the property is new, but should gradually fall to about 60% of the cost to reproduce new when the plant has reached an age greater than the probable life of any of its parts.* If bonds are issued in excess of this amount, and the property is turned over to the bondholder to cancel the obligation, he would find the value of the property inadequate to meet the indebtedness with which it is burdened; the bonds could not maintain a par value.

There are, of course, cases where bond issues are required to replace worn-out parts. In such cases the funds provided by the bond issue restore the particular parts which are replaced to a 100% value, and this full value, and not the remaining value of the old parts, at the time of commencing proceedings for the issue, is the proper basis on which to determine the permissible limit of the issue.

The use of the "replacement" or "depreciation" fund for retiring bonds is a proper use. It makes no difference whether any large surplus in this fund is allowed to accumulate, and is invested in interest-bearing securities, whether it is used in extending the plant, or whether it is used to discharge a debt on which the owner is paying interest, provided only that an adequate portion of the fund is readily convertible when required to pay for replacements.

Special Notes.

It is evident from what has here been presented that any attempt to estimate the amount of amortization allowance which should be made, to keep pace with the theoretic depreciation, is, at its best, only a crude approximation; that, if the plan to be followed is one which is to deal with depreciated value and not with the full investment, care must be exercised, not alone in adopting a method of making capital repayments, but also in determining how much of past earnings can be construed as having been applied, first to amortize early losses, cost of establishing business, and, second, to amortize capital, and in determining therefrom the remaining investment.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 784.

To avoid these uncertainties and to establish a better relation of the earnings of the utility to the ability of the rate-payer to pay, recourse should be had to the method of valuation designated as the Unlimited-Life Method, whenever applicable, thereby eliminating all necessity for close approximations of depreciation. This method has easily recognized advantages, both in the case of old, well-established properties, and in the case of properties hereafter to be established.

In fixing rates, if amortization of capital is determined on, cognizance must be taken of the adopted plan of amortization.

Either (A), the amortization continues during the actual life of the article whether less or greater than its probable life new;

Or (B), the amortization ceases at the end of the probable life term, regardless of whether the article covered by the amortization has failed early or whether it continues in use long after the end of its original probable life term.

These are correct methods of amortizing capital, whether in uniform annual amounts, in increasing annual amounts, or according to any other plan.

When franchises have a definite time limit, at the end of which the plant is to be disposed of to the public, there may be some special provision for the transfer. If the transfer is to be made without any payment at the end of the franchise term, or at any elected time under a franchise with indeterminate life, then the invested capital should be returned during the time that the plant is in private ownership. If, on the other hand, the transfer is made, as it should be, on the basis of an investment by the public in the property at the time it is taken over, then the remaining investment should be determined according to the appraisal method that has been in vogue during the life of the property, and this remaining investment, which may be 100% under the Sinking-Fund and Unlimited-Life Methods, or less than this under other methods, will be the amount which the original owner should receive.

The past history of a public service property should always be taken into account when an appraisal is made which is to serve as a basis for fixing rates. It is unjust to assume that a part of the investment has been amortized unless the records or other satisfactory evidence shows this to have been the fact.

When rate-regulating bodies, such as Public Service Commissions, lay down rules requiring appraisals to be made of depreciated value (particularly when amortization, under the guise of depreciation, is subtracted from cost), grave injustice may result therefrom to the owners of established public service properties, and it is notably in such cases that experts exercise their ingenuity to bolster up values.

There are difficulties all along the line when appraisals are made by any other than that which has here been called the Unlimited-Life Method.

Depreciation is not amortization. It has nothing to do with the repayment of invested capital, and should always be considered apart from amortization.

It is difficult, usually unnecessary, and almost impossible to compute accrued or current depreciation correctly.

It is generally undesirable to amortize the investment in a privately owned public utility.

It is undesirable and yet sometimes necessary to include in appraisals value for intangible elements, such as early losses, or cost of establishing the business, as is frequently done, particularly when depreciated values are used as a basis for determining allowable earnings.

It is undesirable to permit appraisals for rate-making that will cause fluctuations in earnings, except only such as show a gradual increase of aggregate earnings, keeping pace, as time goes on, with the increasing number of rate-payers.

The service rendered should be at all times up to an established standard, and the price of the service should be fairly stable, with a downward rather than an upward tendency.

The bonding value should be as stable as circumstances will permit. This is secured by adopting the Unlimited-Life Method of making appraisals for rate-fixing purposes whenever it is applicable.

The Equal-Annual-Payment Method should be regarded as impracticable, except under inflexible rules making probable life the prime consideration, or requiring that due regard be had to expectancy. In the former case, the amortization will begin for each new investment as made and will continue for the assumed probable life term and no longer. Then, under such a rule, there will be times when amortization will be allowed on some article already out of service

at the same time that the cost of the article which replaces it is being amortized, and there will be other times when the cost of an article is completely amortized, and no amortization installment is to be provided for. Where single articles, or plants made up of very few articles, are under consideration, the undesirability of such a rule is apparent. Under the alternative rule, the method becomes difficult and inconvenient to apply. The Sinking-Fund Method is always to be preferred.

There are cases, as, for example, in a steamboat business with only one steamboat in service, where the advantages of the Sinking-Fund Method, as herein explained, over the other methods of appraisal, are readily apparent.

The Straight-Line Method is generally undesirable, because, if applied from the beginning, it entails early high aggregate earnings which must be collected from only a few rate-payers, and because its application is only correct when it has been in continuous use from the beginning.

The Unlimited-Life Method makes all considerations of depreciation, except for the purpose of tentatively approximating replacement requirements, unnecessary. It reduces the services required of valuation experts and accountants. It should be insisted on by the owner whenever it is applicable, and it should be elected by the rate-payer because it is fair and will make the required earnings least in amount when there are fewest rate-payers, that is, in the early years of operation.

The decisions of the Courts in defining fair value and in declaring for depreciated value are not to be taken as conclusive. They will be modified when these matters are better understood.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

REPORT ON A SERIES OF TESTS ON CONCRETE COLUMNS REINFORCED WITH A SPIRAL OF STEEL.

Discussion.*

By C. G. Wrentmore, M. Am. Soc. C. E.†

Mr. Wrentmore.

C. G. Wrentmore,‡ M. Am. Soc. C. E. (by letter).—The denunciation by Mr. Godfrey of the column reinforced with longitudinal rods is more emphatic than is considered to be justified by either our experience or by the results of analysis, so far as the latter can be applied. These tests were begun more than 7½ years ago, and prior to that time the writer had concluded that for such columns the treatment then current was unsound, and that he would make no further use of them except with thorough hooping.

There are cases, however, where the use of longitudinal rods in members designed primarily for compression is justified. The writer's experience in the Philippines, where high wind pressures and mild earthquake shocks are frequent, and where violent earthquakes may be anticipated, at least in some localities, has demonstrated that reinforced concrete is peculiarly well fitted to withstand the resultant stresses. Although he would use longitudinal rods to resist the bending stresses, they would not be included in designing for direct compression, except where hooped at frequent intervals.

Neither is the writer inclined to concur in the logical conclusion derivable from Mr. Godfrey's statement that "though the Profession has been in possession of this experimental demonstration for a number of years, the facts have been so perverted, and recommendations

* Continued from April, 1914, *Proceedings*.
† Author's closure.
‡ Manila, Philippine Islands.

Mr. Wrentmore.

have been so utterly contrary to what they have shown, that standard methods of constructing columns are criminally bad." It is undoubtedly true that in the past, much work has been done, in reinforced concrete, which is unsound, and that in the future, faulty designs may be drawn; but the possibilities and limitations of reinforced concrete are not to be learned in a day, and the Profession is rather to be congratulated that the history of its use shows so few failures, than to be accused of a lack of either acumen or honesty in the interpretation of experimental data.

As to Conclusion (5) of the paper, that "The limit at which the first visible sign of failure occurs is raised by spiral reinforcement. This effect is greatest in short-time tests, and is perceptible for all percentages of reinforcement": Mr. Godfrey says (after reducing certain loads to unit stresses on the full section of the hooped column, instead of on the core) "if these facts teach anything at all, they blazon out the truth that hooping has no influence whatever on the first failure of a column * * *".

The writer holds that the tests, as given, demonstrate: (*a*) If the sectional area of the hooped column is taken as the effective area, Conclusion (5) is plainly true; (*b*) If the sectional area of the hooped column is taken to include the concrete outside the hooping, the long-time tests show no increase due to hooping, but the short-time tests show an increased strength which is greater with the larger quantities of steel. In Table 8, the writer compares the unit stress at first sign of failure of the columns of the various groups, basing it on the full section of all columns.

TABLE 8.—UNIT STRESS AT FIRST SIGN OF FAILURE.

Group.....	1	2		3		4		5		6	
Percentage of reinforcement.....	0.0	0.44–0.54		0.67–0.86		0.16–1.35		1.41–1.69		2.02–2.32	
Age.	Stress.	Age.	Stress.	Age.	Stress.	Age.	Stress.	Age.	Stress.	Age.	Stress.
40	1 089	35	1 650	46	1 578	45	1 970	51	1 980	42	2 020
68	1 796	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
313	2 763	336	2 495	345	2 490	365	2 781	277	2 980	393	2 100
428	2 903	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Table 8 shows that not only for an age averaging roughly 45 days is the strength increased by hooping, but also that this increase is very considerable, the smallest amount shown being that of Group 3, where it is 45% for from 0.67 to 0.86% of steel. (The 1 650 of Group 2 is given by a single column, and is abnormally high.)

Mr. Wrentmore.

For this age the lowest value for a single column is 1 650 for Group 2; 1 255 for Group 3; 1 330 for Group 4; 1 915 for Group 5; and 1 680 for Group 6; and the minimum for Group 1, plain columns, is 815; so that, if minimum values are taken, the increase is 54% with from 0.67 to 0.86% of steel. Moreover, Mr. Godfrey says:

"Hence the 'first sign of failure' is the function that must govern in safe design. This value, in these tests, runs from 1 400 lb. per sq. in. up, on the full area in compression, on the hooped columns, or 1 800 lb. on the core."

The hooping will be more effective in raising the first sign of failure as the modulus of elasticity decreases, or, more exactly, as the deformation under a given unit load increases. This would result in a greater effect in a weak mixture and also in green concrete, and less in a strong mixture, and would gradually decrease with age. As the tests show that the increase is about 45% at 45 days, it should be expected that for 20 or 30 days, it would be even more.

The fact that economy calls for stripping the forms at an early date, that under actual working conditions they are frequently removed as early as 14 days, thus throwing the load on the concrete while the elastic modulus is still quite low, makes this increase of the limit of first sign of failure especially useful.

The weakness of the concrete at this time is due to the weakness of the cement, and supplementing the bonding action of the cement by the use of a spiral of steel, the strength of which is accurately determinable while that of the green concrete is always questionable, gives the structure an insurance against failure at the time when failures have usually occurred. The large deformation which may occur on stripping forms from a concrete which is slow in hardening, throws the spiral into stress, and so puts it in condition to exercise a greater influence at any later date. Again, the percentage of ultimate strength at which the first sign of failure occurs is lower at the earlier age, and this fact acts as an insurance to workmen, plant, and building against an abrupt collapse without any warning. Such collapse involving an entire structure, may result from weakness in, or overloading, a single column; whereas, if the member, by flaking or spalling, had given warning that failure was impending, much damage might have been averted. It is under precisely these conditions that the column with longitudinal rods is especially undesirable, as the large deformation of green concrete induces high compressive stresses in the steel which tend to cause buckling, so that the steel may actually weaken rather than strengthen the column, unless the rods are hooped at frequent intervals.

The relation of ultimate strength to first sign of failure is of interest only in so far as it may prove to be an insurance against loss of life or property by giving warning of imminent failure. No

Mr. Wrentmore.

engineer can be sure that his structure will not at some time, through ignorance or carelessness, be subjected to loads in excess of those for which it was designed, and it is not undesirable that in such case the structure be capable of giving warning of its overstressed condition rather than to fail abruptly.

Mr. Buel's inquiry regarding the method of determining the value of E is answered fully in the paper. An examination of Fig. 17 will suffice.

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PAPERS AND DISCUSSIONS

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SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

Discussion.*

By Robert E. Horton, M. Am. Soc. C. E.

Mr. Horton.

Robert E. Horton,† M. Am. Soc. C. E. (by letter).—This paper presents an argument in favor of the extension of the doctrine of priority in the use of streams, as now adopted, to some extent at least, in the Western United States and other irrigation countries, so as to include in this practice the eastern part of the United States, where the doctrine of riparian rights generally prevails. The writer feels obliged to take exception to some of the author's arguments and conclusions; at the same time, it is recognized that there is vast need for improvement in riparian law and practice regarding water rights in the Eastern United States. How this may best be brought about is a problem yet to be solved. In one thing the writer agrees most heartily with Mr. Johnston, and that is that some system should be devised by which the tedious and expensive litigations now prevalent, both in the East and West, can largely be obviated. The writer also agrees with the author (and believes that most engineers in the East who have had wide experience in water litigation will also concur in the opinion) that many of the so-called judicial questions arising in water-right decisions could be settled far better by engineers, under certain judicial restrictions, than as they are at present by the Courts. The writer does not mean to cast any reflection on either the judicial or legal fraternity; perhaps the blame rests as much with the Engineering Profession as anywhere; but it is a fact that many water-right litigations resolve themselves, in simple language, into a war of experts, decided by a referee who is not competent, through

* Continued from October, 1914, *Proceedings*.
† Albany, N. Y.

Mr. Horton.

knowledge, training, or experience, to judge exactly the merits of the contentions of either side. It has happened a number of times in the writer's experience that after a tedious and costly litigation over questions of water rights, a decision has been handed down by an entirely impartial judge or referee—a man of the highest ability from a legal standpoint—which was almost wholly incapable of practical application and required another lawsuit to decide the decision.

Looked at from an Easterner's viewpoint, it seems to the writer that the doctrine of priority, as applied to water rights, is fully as pernicious as the doctrine practiced in the East appears to the author. If the doctrine of priority is good, then the argument advanced by Mr. Johnston, that municipalities should be given the right to divert water from streams without compensation to vested power or other riparian interests, does not appear to be sound. The power interests are generally there first. If priority of use is a sound basis for establishment of right, then their rights are established thereby, and they should not be dispossessed without remuneration. The writer believes that in many instances vested interests have grown up in a manner inimical to public interest, and that curbing and regulating the use of public waters is necessary, but it must not be forgotten that the upbuilding of industrial welfare in the East is in a considerable degree the result of the utilization of water power by individual enterprise in early times, when land and water rights were unsalable or of little value.

The public has received, through taxes and other indirect benefits, enormous remuneration for the utilization of these rights, and, in the majority of instances, the writer fails to see where there is any large element of unearned increment in value appertaining to the occupant or reputed owner of such rights.

Municipal governments are notoriously lacking in foresight in securing adequate sources of water supply. Suppose a city has grown to a population of 100 000 during the past century, has outgrown its existing sources of water supply, and must now take a new source from some large stream which, heretofore, has been developed extensively for power purposes. The writer cannot see any element of justice in a doctrine which would permit the municipality at this late date to seize without remuneration holdings on this stream, and divert the waters to its own uses. Had the city administration a century ago exercised the same foresight which was exercised by private enterprise in acquiring rights on the stream, then the present necessity for acquiring such rights after their intensive development by private enterprise would not exist. On the other hand, excessive awards undoubtedly have often been made in cases of municipal diversion from power owners. Legal barriers and obstructions have been placed in the way of a municipality seeking to secure better water

supplies. To some extent public health has no doubt been jeopardized thereby for longer or shorter periods. This condition is primarily chargeable to the fact that all humanity is more or less human, and, being so, is greedy, grasping, and tenacious of its holdings; but, at the same time, it is believable that vast improvements could be made in methods of adjusting differences between municipalities seeking to divert water, and the owners of power interests affected thereby. The author's suggestion, that this can be accomplished through the utilization of engineers as arbitrators, with certain judicial functions, seems to be a good one. This method has not infrequently been tried in the East, and, so far as the writer has observed, the results have generally been more satisfactory, more promptly attained, and far less costly than when the usual condemnation proceedings have been followed. Mr. Horton.

The principal sources of litigation regarding water power, riparian rights, and diversion, in the East, as they have come under the writer's observation, are as follows:

1.—Land titles.

a.—Questions as to boundaries of grants on and along streams: whether they run to the bank or center of the stream, etc.;

b.—Questions of interpretation of terms used in land boundaries along streams, such as the "thread of the stream", "low-water mark", "ordinary stage", "high-water mark", etc.

2.—Flowage.

a.—Back-water from one dam affecting up-stream interests within the stream channel;

b.—Flooding of lands adjoining streams.

3.—Diversion.

a.—By municipalities, as against water-power and riparian owners;

b.—As between owners on opposite banks of the stream;

c.—As between different owners on the same dam or power canal.

4.—Interpretation of grants.

Different classes of cases arising will be discussed later.

5.—Priorities and surplus rights.

6.—Prescription, abandonment, and unlicensed use.

7.—Transference of riparian rights, change in character of use, etc.

8.—Pondage and storage, or other interference with the natural regimen of the stream.

In the writer's opinion, litigations arising out of the interpretation of grants would not be mitigated in any way by the adoption at the present time of the doctrine of priorities.

Mr. Horton.

The writer believes that existing grants, made years ago and indefinite in character, cannot now be reduced to simple terms, except by the tedious processes of litigation or by arbitration by a board of competent engineers. As illustrating the kind of questions arising from such grants, the usual language in which water was granted in the early days for power purposes at Eastern mill seats is briefly analyzed as follows:

1.—Enough Water for a Specific Purpose.—Grants of this kind have various degrees of indefiniteness, as, for example, quoting from the writer's experience: "Enough for a forge shop"; "enough for a saw-mill"; "enough for a woolen mill 20 ft. by 30 ft."; "enough to drive all the machinery which can be economically used in a cotton or woolen mill four stories high"; "enough for a fulling mill and oil mill, a carding mill, and enough to run a pump." The most common form of such grants was the use of the expression "enough water to drive one run of millstone." Grants of this kind were more or less indefinite. In some cases, the size and speed of the stone, kind of grain, and number of bushels to be ground per hour, were all stated. Such grants were capable of very definite interpretation. In many instances it is specified in such a grant that it conveys also enough water to supply power for the necessary machinery appurtenant thereto. In the majority of cases all that is stated is that the grant conveys enough water for a run of millstone, although the most common qualifications are the size of the stone, and whether or not power for attendant machinery is included.

2.—Enough Water for a Specified Water-Wheel.—Water rights in this language arise:

a.—From original grants;
b.—From prescription;
c.—From later interpretations of early indefinite grants.

Grants in terms of water enough for a specified water-wheel, if the wheel is a turbine of recent type and standard construction, are fairly capable of accurate interpretation. Unfortunately, however, many cases arise where the interpretation is difficult, as, for example, the grant of "enough water for a 40-in. Johnson re-acting turbine wheel as same was constructed in 1842." There were several types of Johnson wheels in use at about that time, and not all of them are the inventions of the same Johnson. Another grant which produced a litigation covering fifteen years, conveyed "enough water to drive three 6-ft. central discharge water-wheels." Grants conveying enough water for an overshot water-wheel 10 ft. in diameter, length not specified, and the like, are fairly common.

3.—Grants in Terms of Definite Fraction of the Flow of a Stream or Canal.—These are capable of interpretation if the capacity of the

canal or the economical basis of power development in the stream are determined. Mr. Horton.

4.—Grants in Terms of a Certain Number of Square Inches.—In order to distinguish this term from miner's inches, it may be defined as a certain number of millwright's inches. This term was applied to the vent or capacity of water-wheels, and, when thus used, invariably meant a stream of water of the actual area specified, flowing with a theoretical velocity due to the head. However, when the term came to be applied to the definition of water rights, openings in flumes, etc., disputes arose as to whether it meant actual square inches of stream or square inches of opening, that is, whether or not a coefficient of contraction was to be applied. This question has been the source of many large and costly litigations.

5.—Enough Water to Fill a Certain Pipe.—Efforts to interpret grants of this kind illustrate the necessity for technical or hydraulic knowledge on the part of those intrusted with their interpretation. This was forcibly brought home, in one instance, by the engineer being asked by the Court how large a pipe a certain quantity of water would fill, after witness had stated the quantity of water to be diverted from a stream in terms of gallons per 24 hours.

6.—Grants in Terms of the Right to Take the Flow Through an Opening of Specific Dimensions.—Grants of this kind are productive of litigation as to form of opening, question of contraction, etc.

It is rare to find a water grant in the East in terms of cubic feet per second, or cubic feet per minute, or other definite language. As illustrating apparently the maximum degree of indefiniteness capable of being promulgated in a grant of land and water, the following approximate quotation is made from a deed of a water lot in Northern New York, a copy of which is in the writer's possession. The deed describes a parcel of land granted by bearings and distances, and ends thus: "thence S. 28° W. 10 chs. 44 lks. to the southwest corner of the house now occupied by the widow Hawkins, be the same more or less." This deed was drawn by an old-time country surveyor and engineer, subject, apparently, to lapse of memory under certain trying circumstances, whereby he omitted to specify the acreage in this case.

As a matter of curiosity, the writer compiled the principal causes involved in some fifty water cases within his experience. The results are as follows: back-water, 8 cases; interpretation of grants, 8 cases; diversion, 14 cases; appropriation, 17 cases; and other causes, 3 cases.

Diversion and appropriation should properly be classed together; they comprise 60% of the total. This suggests forcibly the necessity of better methods in handling diversion cases than those now prevailing.

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PAPERS AND DISCUSSIONS

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SUBAQUEOUS HIGHWAY TUNNELS.

Discussion.*

By Duncan D. McBean, Esq.

Mr. McBean.

Duncan D. McBean,† Esq.—Mr. Snyder gives a cross-section of the Harlem River Tunnel (Fig. 24), and sections showing the working platforms (Fig. 25), which, for convenience, were used instead of scows in constructing the subaqueous working chamber in which the tunnel was built. In discussing the method of constructing the tunnel, Mr. Snyder writes of it as being "a variation of the coffer-dam method", which is not only a misnomer, but also misleading. The patent obtained by the writer for this method is for a "Subaqueous Working Chamber"; he believes, therefore, that the method should be designated the Subaqueous Working Chamber method.

The advantages of this method are not only those accredited to the Detroit River method by Mr. Snyder, that is:

"That a structure can be placed at a higher level than with a driven tunnel, with a consequent reduction in the gradients of approach, and the section can be given a shape conforming to the use to which the tunnel is to be put, without being limited to the circular section usual with a shield";

but it also has many other advantages, some of which are as follows:

In cases where the circular form is desirable, instead of having a metal shell completely encircling each tube, it permits of joining the upper and lower segments of the metal shells to a vertical partition wall between them, thereby forming a twin-tube tunnel, and very materially reducing the quantity of materials required and likewise greatly reducing the surface of metal shells exposed to hydrostatic

* Continued from October, 1914, *Proceedings*.
† New York City.

Mr. McBean.

pressure; it enables the excavation to be made to the neat width and depth required for the structure; and it permits of placing all the masonry, both outside and inside the metal shells, in air.

The water-proofing can be placed on the outside of the masonry, thereby permitting the utilization of all the structural masonry to withstand the surrounding hydrostatic pressure, and thereby greatly diminishing the quantity of material necessary to overcome the buoyancy of the structure during construction.

Mr. Snyder shows a cross-section of the Detroit River Tunnel (Fig. 28), but it is submitted that his conclusion, that

"The method used in constructing the Detroit River Tunnel has been described so thoroughly in the paper by W. S. Kinnear, M. Am. Soc. C. E., and in the paper by W. J. Wilgus, M. Am. Soc. C. E., that but little comment is necessary,"

finds scant support in the papers* referred to.

An analytic examination of the several papers describing the construction of the Detroit River Tunnel shows: That sections of concrete forms consisting of the metal shells for inner forms, with vertical walls of sheeting attached thereto for outside forms (which, taken together, may properly be called sectional skeleton frames of a subaqueous working chamber), were lowered into the dredged trench and rested on temporary supports under the ends of the sections.† These temporary supports consisted of grillages supported by piles, or as Mr. Kinnear calls them,‡ "a 10 by 10-in. timber leg, or spud, about 10 ft. long, which was driven into the clay by an ordinary pile-driver drop-hammer * * *". Under the skeleton frame when resting on its temporary supports, the distance to the bottom of the trench varied "from 1 to 6 ft., according to the depth of trench left by the dredge."§ The foundation concrete under the skeleton frame was deposited through tremie pipes in the water on the muddy bottom of the trench. In order to use tremie pipes for depositing the concrete in the water under the structure, it was necessary to place the tubes 3 ft. apart and to encircle each tube completely with a metal shell, thereby not only causing the track centers in the completed tunnel to be 26 ft. 4 in. apart (in the Harlem River Tunnel the distance between the track centers is only 12 ft. 6 in.), but also increasing the buoyancy, to overcome which necessitated the placing of a large surplus of concrete on the outside of the metal shells. The skeleton frame around the outside of the metal shells was then filled with concrete through tremies, thereby completing the side-walls and roof of a subaqueous working chamber. The water within the metal shells

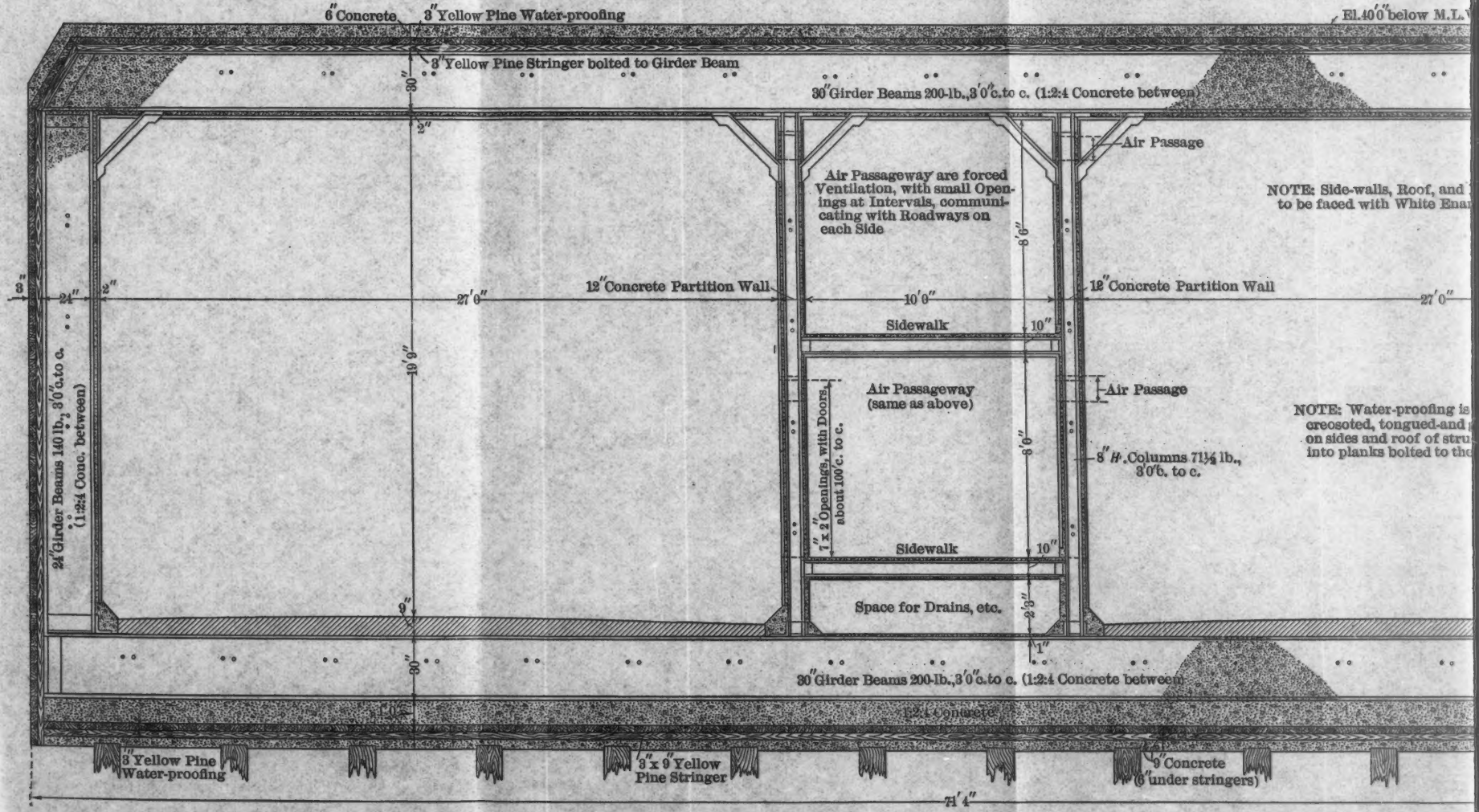
* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 288; and *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXV, p. 2.

† *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 330.

‡ *Ibid.*, p. 331.

§ *Ibid.*, p. 338.

DESIGN FOR ROADWAY TUNNEL UNDER DETROIT RIVER,
BETWEEN DETROIT, MICHIGAN AND WINDSOR, ONTARIO,
IN COMMEMORATION OF 100 YEARS PEACE BETWEEN UNITED STATES AND GREAT BRITAIN



DESIGN FOR ROADWAY TUNNEL UNDER DETROIT RIVER,
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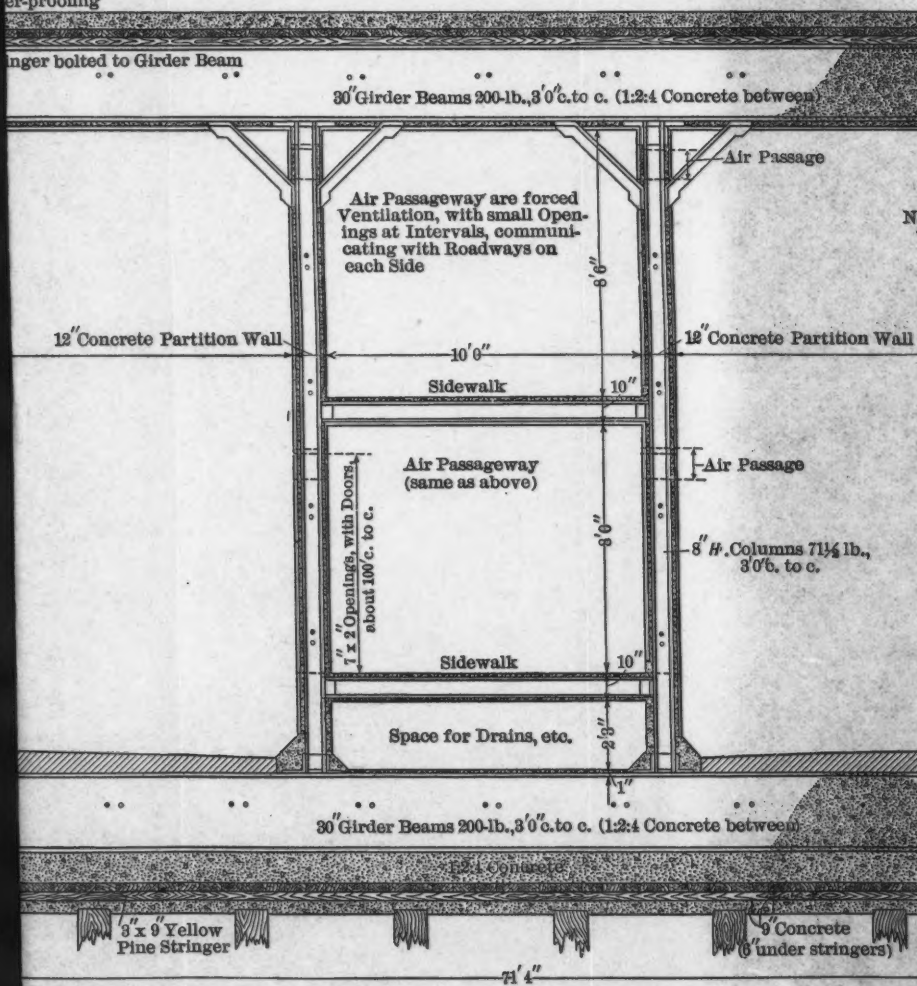

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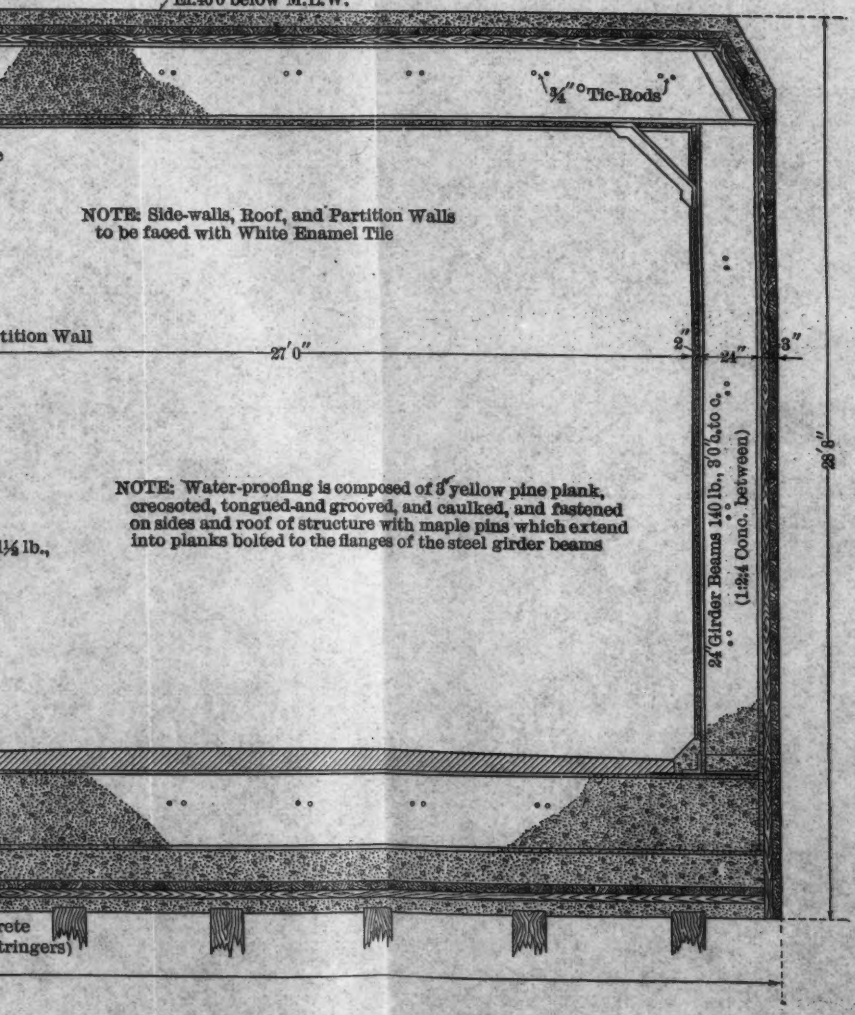

was then pumped out, resulting in the completion of a pneumatic subaqueous working chamber, within which the "tunnel proper" was built.* Mr. McBean.

Mr. Kinnear states that in the fall of 1904, a committee was appointed to report on the feasibility of a tunnel under the Detroit River, at Detroit, and that the report was to include estimates of cost, etc. He states that the committee made the necessary investigations, reported favorably on the construction of a tunnel, and submitted estimates covering the different gradients for approaches, recommending a 2% grade west bound and a 1½% grade east bound. That the estimate as then prepared for the recommended tunnel was a good one, has been demonstrated by the actual cost of construction.

He likewise informs us that in July, 1905, the Committee was superseded by an Advisory Board of Engineers, and that the work of preparing plans was started in August, 1905, but he fails to mention the source of the Committee's data on which it based its estimate of the cost for the construction of the tunnel by the Subaqueous Working Chamber method, concerning which Mr. Hoff writes:

"effected a saving of about $2 000 000 over a shield-driven tunnel with compressed air, and to this should be added the capitalized saving in annual cost of operation on account of the tunnel being placed some 15 ft. higher, thus reducing by this amount the vertical lift of the tonnage passing through."†

Mr. Snyder's Fig. 27 is a cross-section of a roadway tunnel, which the writer prepared and submitted to the Hoboken Board of Trade. Since then he has prepared and submitted a design (Fig. 34) for a roadway tunnel under the Hudson River, on the proposed Canal Street Line, accompanied with a proposal to build the tunnel and its approaches for the sum of $5 500 000—just one-half the estimated cost of building a circular tunnel with roadways of the same width by the shield method. The writer has also prepared and submitted a design (Plate XXXIX) for a proposed highway tunnel under the Detroit River, at Detroit, Mich. The number of piles under the latter structure is greater than would be required ordinarily, except in places where the bottom of the river is filled in for the purpose of supporting the tunnel.

It was the writer's conception to build subaqueous tunnels, according to these designs, by constructing long sections of the tunnels in pontoons in which all the structural material would be put in place, except the concrete between the bottom girders inside of the sidewalls; bulkheading the ends of the sections; attaching either to the top or to the underside of the bottom girders a temporary water-tight floor; sinking the pontoon and removing it from under the struc-

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, pp. 303, 328, 362.
† *Ibid.*, p. 371.

Mr. McBean.

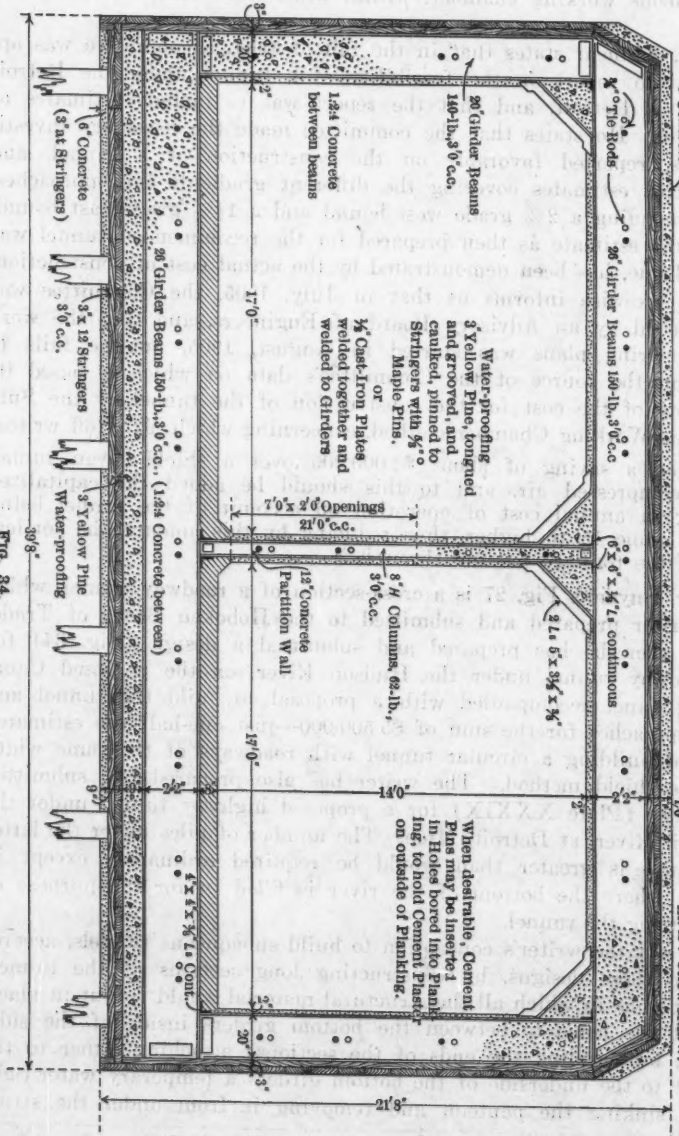

FIG. 34.

ture; overcoming the buoyancy of the structure by placing within it on its temporary floor a part of the material required for the bottom concrete; then lowering the structure on a temporary foundation of piles, driven into the bottom of the dredged trench, and cut off and capped at the proper depth to support some of the bottom girders; admitting a pressure of air, equal to the hydrostatic pressure at the level of the temporary floor; and then removing the temporary floor. The structure so constructed would then form a working chamber. The foundation would then be cleaned out, the foundation concrete, water-proofed with planking, would be put in place in air, and the section of tunnel completed. Mr. McBean.

The rectangular form of these designs, together with the water-proofing of creosoted, tongued-and-grooved, yellow pine planking, pinned on the outside of the structural material, renders them simple, durable, and economical for future subaqueous tunnel construction.

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THE STEREOSCOPIC METHOD OF SURVEYING, AND A FIRST TRIAL OF ITS APPLICATION TO A RAILWAY SURVEY IN CHINA.

Discussion.*

BY F. LAVIS, M. AM. SOC. C. E.

Mr. Lavis.

F. LAVIS,† M. AM. SOC. C. E. (by letter).—This description of the application of the methods of stereoscopic surveying to railroad location is interesting from a theoretical standpoint, but there is little in the paper to show its value commercially, that is, to show that the necessary information for the location of the best line for the purpose of railroad construction through this valley was obtained by this method, at less cost than would have been possible by ordinary methods. As a matter of fact, cost is hardly mentioned except to note that one of the desirable minor instrumental aids costs about $6 000.

Apparently 10 men (engineers or engineering students) consumed most of 2 months in the necessary field work covering from 4 to 6 miles of line, though this is not quite clearly stated; if so, it hardly seems as if this method would compare favorably with those ordinarily used, namely, the typical American method, or the stadia method more generally used by European engineers.

There seems to be little in the paper to indicate the final degree of accuracy obtained. Speaking generally, it would seem that a map on a scale of 1:2 500, or approximately 200 ft. to the inch, which is virtually an enlargement from a very much smaller scale photograph, would have a rather large probability of error. As the matter stands,

* This discussion (of the paper by Georg A. G. Müller, Esq., published in August, 1914, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Lavis.

it would almost seem as if the topographical map produced by this method was of approximately the same value as the topographical sheets issued by the United States Geological Survey on a scale of 1:62 500, which are excellent for reconnaissance projections, but not for anything more detailed; perhaps, however, a further explanation by the author may throw more light on this. Of course, assuming adequate results, the true test of any survey method for the location of a railway line is the cost of the final line staked out on the ground. It must always be kept in mind that the ultimate purpose of a railroad location survey is not the making of a topographical survey or the production of a topographical map, but the location on the ground of the best line the country affords under certain given conditions, and incidentally doing the work by methods which involve the least cost. The production of a topographical map, if this is necessary or even desirable, as it almost always is, is merely incidental to the final purpose, and its extent and accuracy should be governed solely by its use for the ultimate purpose in view.

It seems to the writer that, even disregarding the first cost of the very expensive outfit, and assuming that the stereoscopic method might enable one versed in its use to produce at a reasonable cost a topographical map having the necessary degree of accuracy to enable a line to be properly projected on it, the cost of actually reproducing this line on the ground would be considerably more by this process than by the usual American methods.*

It may be thought that, in using the stereoscopic method, the relation of the projected line to the triangulation points might be determined from the map and the line staked out from this, but this seems to the writer to be likely to produce results very much inferior to those obtained from the reproduction of a line quite close to a previously staked-out preliminary, inasmuch as the work involved in detailed adjustment of the line to the ground in such very rough country would be greatly increased. If the author can give any further information in regard to the final application of this method to the production of a railway line staked out on the ground, rather than the production of a topographical map, which is, of course, all this paper now does, it would be of considerable interest and add greatly to our ability to estimate the value of the new method, and judge of its practicability.

* *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 104; and Vol. LXV (1909), p. 105.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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A METHOD OF DETERMINING STORM-WATER RUN-OFF.

Discussion.*

By Messrs. W. W. Horner, Charles W. Sherman, Robert E. Horton, and C. E. Grunsky.

Mr. Horner.

W. W. Horner,† Assoc. M. Am. Soc. C. E. (by letter).—The author presents a formula for storm-water run-off and introduces it as "empirical." He speaks of the "so-called rational method" and of the "formula method", and states "that the proposed formula gives results in all cases reasonably close to those which will be obtained by the rational method, and that the formula can be taken as an acceptable equivalent." Elsewhere, he speaks of the rational method as involving "cut-and-try methods" and as being "relatively laborious."

The author does not include in his paper any information in regard to the development or construction of his formula. The writer became interested in the manner of appearance of the variables in the new formula, and found that, for an ideal condition of area and slope, and with the type of rainfall curve which the author uses in comparing gaugings, he was able to derive from the theory of the "rational method" a formula very similar to that of the author. For this reason, the writer is of the opinion that the new formula should be described as a rational formula for ideal conditions.

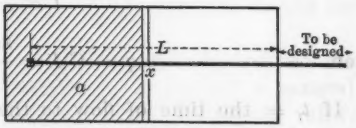

Fig. 5.

The derivation mentioned is of considerable interest, and is presented as follows: Assume conditions as shown in Fig. 5. A circular sewer having a grade of S feet per thousand, a length, L, and a drainage area at the lower end of A. Let a equal the area drained

* Continued from October, 1914, *Proceedings*.

† St. Louis, Mo.

Mr. Horner.

at any point, x, the sewer to be uniformly proportioned to carry the run-off at all points, and the tributary area to be proportional to the length; then the run-off at x will be

$$Q_x = a\,I\,c = 4\,r^2\,B\sqrt{r\,\frac{S}{1\,000}},$$

where r is the hydraulic radius at that point, and B is the coefficient of the Chezy formula.

Eliminating r from these two gives

$$V_x = \frac{B^{\frac{4}{5}}\,S^{\frac{2}{5}}}{26.4}\,(a\,I\,c)^{\frac{1}{5}}.$$

The time of flow may be expressed from

$$d\,t = \frac{d\,(L)}{V_x},$$

or, where

$$\frac{d\,(L)}{d\,(a)} = \frac{L}{A},$$

$$d\,t = \frac{L}{A\,V_x}\,d\,(a).$$

Substituting V_x from above, integrating between $a = 0$ and $a = A$, and dividing by 60 to reduce to minutes:

$$t_L = \frac{0.55\,L}{(C\,I)^{\frac{1}{5}}\,B^{\frac{4}{5}}\,\sqrt[5]{S^2\,A}} = \frac{0.55\,L}{q^{\frac{1}{5}}\,B^{\frac{4}{5}}\sqrt[5]{S^2\,A}}$$

assuming a rainfall curve of the form

$$I = \frac{90\,K}{t + 24},$$

then

$$q = C\,I = \frac{80\,K\,C}{t + 24}.$$

If t_l = the time of flow to the inlet and t the time through the sewer, then $t = t_l + t_L$, or

$$q = \frac{80\,K\,C}{t_l + t_L + 24}.$$

Substituting the value of t_L above and solving for q,

$$q + q^{\frac{4}{5}}\,\frac{L}{1.82\,(24 + t_l)\,B^{\frac{4}{5}}\,\sqrt[5]{S^2\,A}} = \frac{80\,K\,C}{24 + t_l}.$$

The velocity of flow to the inlet may be assumed to be proportional to the square root of the surface slope, or $V_l = x\sqrt{m}$.

If l = an average distance from the water-shed to the inlet, then

$$t_l = \frac{l}{x\sqrt{m}}.$$

Mr. Horner.

For average conditions, assume $l = 500$, $m = 10$, and $t = 5$.

Then $$\frac{l}{x} = 15.5, \text{ say } 16,$$

and $$t_l = \frac{16}{\sqrt{m}}.$$

Also, assume an average value of the Chezy coefficient, B, as 110; then the formula becomes

$$q + q^{\frac{4}{5}} \frac{L}{78 \sqrt[5]{S^2 A} \left(24 + \frac{16}{\sqrt{m}}\right)} = \frac{3.33 \sqrt{m}\ K\ C}{\sqrt{m} + 0.67}.$$

This formula, though based on several averages and assumptions, is a rational one; it differs slightly from the author's formula in the numerical coefficient and in the appearance of C under the radical with $S^2 A$. This appearance of C is irrational, and may warrant the use of the word empirical as describing the formula. The term, C, appearing as a one-fifth power, however, has little effect on the results, because, for a range of C from 0.3 to 0.8, $C^{\frac{1}{5}}$ varies only from 0.79 to 0.96.

If an average value of 0.9 were introduced into this formula, it would become

$$q + q^{\frac{4}{5}} \frac{L}{86 \sqrt[5]{C S^2 A} \left(24 + \frac{16}{\sqrt{m}}\right)} = \frac{3.33 \sqrt{m}\ K\ C}{\sqrt{m} + 0.67},$$

which is almost identical with that of the author. If other values of B, C, and t_l had been assumed, the numerical coefficients would not have agreed, but the manner of appearance of the variables would have been the same.

The writer wishes to ask why the coefficient, C, is called the ratio of "run-off to total rainfall"? The coefficient of run-off is generally considered as the ratio of the rate of run-off to the intensity of rainfall producing that rate. This latter definition has a definite meaning in sewer work, and the effect of changes in it are readily understood. The C, as defined by the author, is of importance in water-supply work, but does not appear to have any known relation to the rate of run-off.

Whatever the origin of the formula, it may be seen from the foregoing derivation that it may be expected to give reasonable results for certain average conditions. The conditions met in actual practice, however, will vary widely from any mean. In the main sewer of any large system, frequent changes of size and grade, of surface conditions, and even of soil conditions, will be encountered. By the rational method, all these items will enter in their proper place and be taken into account; in any formula, only a general mean condition can be considered, and, in any particular case, the results may vary widely.

Mr. Horner.

The writer is of the opinion that the formula will not give results in all cases reasonably close to those obtained by the rational procedure, and that in general it will not effect any economy of labor in design. In the new formula, all the data and the same analysis of them are required as for the rational method. For the purposes of design, a sewer is divided into a number of sections, according to the areas drained and the grades. In the rational procedure, the grade of a section enters into the calculation of the time of flow for the section, and has no relation to the grades of other sections; in applying the formula to each section, an average of all grades above is required. Again, the variable, m, in the rational method, affects only the initial inlet time, but in the formula it enters into each solution.

The writer has found the procedure of design by the rational method simple, direct, and comprehensible at every step. Although its application to a single unit of sewer in undeveloped territory may call for cut-and-try methods, in the design of a system of sewers, the procedure is definite, the design of each unit leading directly to the next. This is shown clearly in the tabulation for St. Louis conditions referred to by the author*.

The writer does not wish to discount the value of the new formula as a formula, for it is undoubtedly a great advance over the older ones cited in the paper, but wishes to emphasize his belief that the practice in sewer design has already advanced beyond the stage in which the whole problem may be solved satisfactorily by a single mathematical expression of any kind.

Mr. Sherman.

CHARLES W. SHERMAN,† M. AM. SOC. C. E.—The speaker agrees fully with Mr. Buerger's statement, "It is almost everywhere conceded that the rational system * * *, if used with proper discretion, will give very satisfactory results", but he is inclined to different views on many of the other statements in the paper. He believes that Mr. Buerger is mistaken as to the methods of determining storm-water run-off which are receiving most favor in the larger and better organized engineering departments, and he feels that, in presenting another empirical formula, unless it shall give results very closely paralleling those of the rational method (the possibility of which the speaker doubts), he is tending to foster a method of design which is irrational.

The author suggests as one of the reasons that, in his opinion, an empirical formula is likely to be more popular in such design than the direct application of the rational method, that the latter is relatively laborious, and "requires, in addition, a material exercise of judgment." With regard to the comparison of the labor required in the solution of the true rational method and the use of the suggested

* *Engineering News*, 1910, Vol. 64, p. 326.

† Boston, Mass.

Mr. Sherman.

formula, it is evident that the same preliminary work of determining drainage areas, laying out sewer lines, estimating grades, etc., must be done, in any event; and an examination of the diagrams presented with the paper indicates at least a possibility that finding a value of P by following out four lines in one complex diagram, and a value of N by similarly following five lines in another and more complex diagram, and finally, by determining the location of these two values on a third diagram, to obtain the required run-off figure, would be very nearly as laborious as the direct solution of the rational method, besides being accompanied with a greater likelihood of error caused by misreading the diagrams. Disregarding this point, however, does the author wish us to believe that an engineer, not sufficiently familiar with the territory under consideration and with methods of design to be trusted to exercise his judgment in the use of the rational method, can proceed to design storm-water sewers by the use of the proposed formula and diagrams? The speaker is of the opinion that no one without ample understanding of the subject should be entrusted with matters of such importance, and that putting a formula or diagrams in the hands of inexperienced or untrained men and leaving them to obtain results by substitution of values is not to be commended.

Although the author does not claim definitely that the comparison of his formula with the older empirical formulas, such as that of McMath, has any bearing on the adequacy of a method which is intended to give figures closely paralleling those of the rational method, he submits such comparisons in such a way as to lead to their use as a gauge of his formula. The speaker does not think that this comparison should be taken as significant, and, accordingly, he feels that the presentation of the older empirical formulas is out of place in this paper. He would call attention to the fact, however, that the Hering formula, representing the New York diagrams, as quoted in the paper (presumably from the paper* by Charles E. Gregory, Assoc. M. Am. Soc. C. E.), differs from the formula as first written by Mr. Hering, in the Report of the Baltimore Sewerage Commission, 1897, in that the exponent of A is written 0.167, whereas, as given by Mr. Hering, this exponent is 0.15. Although this difference seems slight, a comparison of results obtained by using the two forms of the formula shows that they may differ as much as 15% for large areas.

The author quotes the fact that in storm-sewer design in several large cities, widely different values of R (representing the rate of rainfall) have been used, as an indication that the earlier empirical formulas have been improperly applied. His criticism would seem to imply that R in these formulas was intended to represent the rate of rainfall for a storm of some particular duration. As a matter of fact, however, the use of $R = 4$, in one locality, and $R = 1$, in

* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 458.

Mr. Sherman.

another, merely means that, whereas the latter designer has assumed that the critical storm for which he must make provision is one lasting an hour or more, the former has assumed a condition in which a storm lasting perhaps from 10 to 20 min. will give him the most serious conditions.

If, as Mr. Buerger says, in the application of the older formulas, "it is likely that, for a mountainous, rocky district of small size, the same constants may be used as for a flat beach town of large extent", what is to prevent a similar misuse of constants in his proposed formula?

Mr. Buerger's statement, "that formula methods are much more popular", may be true if tested by the number of individual engineers using one method or the other. On this point the speaker has no information, but he is of the opinion that present practice in the engineering departments of the larger American cities, and some of the smaller ones where considerable attention has been paid to this subject, is distinctly in favor of the use of the rational method. Inquiry of the officials of seventeen such places has shown that ten of them use the rational method or some of its modifications, two use the McMath, three the Bürkli-Ziegler, and two the Hering formula. The places referred to are as follows:

METHODS USED IN ESTIMATING STORM-WATER RUN-OFF IN VARIOUS CITIES.

The Rational Method and its Modifications.—Ten places.

New York City, Borough of Queens.
" " " " " Richmond.
" " " " " The Bronx.
Boston, Mass.
Baltimore, Md.
Cleveland, Ohio.
Cincinnati, Ohio.
St. Louis, Mo.
Pawtucket, R. I.
San Francisco, Cal.

The McMath Formula.—Two places.

New York City, Borough of Brooklyn.
Louisville, Ky.

The Bürkli-Ziegler Formula.—Three places.

New Orleans, La.
Worcester, Mass.
Cambridge, Mass.

The Hering Formula.—Two places.

New York City, Borough of Manhattan.
Newark, N. J.

Mr. Sherman.

The speaker also differs with Mr. Buerger's apparent assumption that the results obtained by the application of his formula may be fairly tested by comparison with the results of such sewer gaugings of storm flow as are available, because by far the greater number of these gaugings have been made when the storms were of much less magnitude than those on which design must be based. It must also be borne in mind that in design it is of the first importance that the designer form a reasonable judgment of the future condition of the district under consideration, and adopt his coefficients and constants so that the drains may be adequate when the district has developed, rather than at the present moment, except in the rare case of the district which is already fully developed. It appears, however, that in the comparisons made by the author, his formula gave figures less than the actual gaugings in a large percentage of the cases. It appears, also, that, in making his comparisons, he has omitted many reported gaugings. The Report of the Committee of the Boston Society of Civil Engineers, on Run-Off from Sewered Areas*, contains figures for many more gaugings than those quoted by Mr. Buerger.

In discussing the rational method, the author states that the most complete and satisfactory treatment of it is that given by Mr. August Frühling. Mr. Frühling's analysis is unquestionably complete, and should be studied carefully by every sewer designer. In the speaker's opinion, however, the treatment of the problem given by him is needlessly complex. According to Frühling's analysis, the coefficient, C, in the rational formula, $Q = C\ I\ A$, is made up of three parts: first, a coefficient of rainfall distribution; second, a coefficient of retardation due to the fact that under ordinary conditions all portions of the drainage area are not contributing water at the same rate; and third, a coefficient of retention or of imperviousness, depending mainly on the porosity and roughness of the surface. A clear conception of the effect of these various factors is of the greatest importance in studying the results of gaugings, particularly as many such gaugings have been made when the duration of the heavy rain has been less than the time of concentration of the drainage area, so that the coefficient of retardation is an important factor. In design, however, it seems to the speaker that the most severe conditions must be assumed, in which case the coefficients of rainfall distribution and of retardation are unity, and the run-off coefficient is equal to the coefficient of imperviousness.

The proposed formula seems to be open to serious criticism as to the method in which the rainfall factor is introduced. Mr. Buerger apparently implies that records of intense storms, lasting 60 min., may often be obtained when trustworthy records of intensity of precipitation for shorter periods are not to be had. As a matter

Journal, Boston Soc. Civ. Engrs., June, 1914.

Mr. Sherman.

of fact, the reverse is more likely to be the case. Intense storms of short duration are much more frequent than those of longer duration, and in case the rainfall record covers but a limited period, it is much more probable that it includes figures approximating maximum or critical conditions for periods of less than ½ hour than for longer periods. It is extremely unsafe, therefore, to infer from any limited record of rainfalls for periods of 1 hour what the intensity of precipitation may be which should govern storm-sewer design.

Even were it possible to determine readily what rate of precipitation for a period of 60 min. is to be expected or assumed as the critical rate, for the purposes of design, it would still be improper to assume that the intensity for any other period of time bears some definite ratio to that for 60 min., as seems to be the case in the proposed formula. The only statement in the paper relative to this ratio, which the speaker has been able to discover, is that at the foot of page 1742,* where it is assumed that "$I = \frac{80\,K}{20 + t}$", or, in other terms, $\frac{I}{K} = \frac{80}{20 + t}$. This relation holds true only for a rainfall curve of the form $I = \frac{A}{20 + t}$. Although this curve has been assumed by some designers as representing approximately rainfall conditions in the Eastern Atlantic States, others have derived altogether different curves, and this form would not be at all applicable to some other parts of the country.

The speaker has had occasion recently to study rainfall records in a number of the more important cities throughout the United States, and to bring together rainfall curves suggested by various engineers for different localities. Though the form indicated by the formula, $I = \frac{A}{B + t}$, is popular with a large number of engineers, for some localities values of B widely differing from 20 have been found necessary to fit the conditions discovered, and other engineers have used formulas of the exponential type, represented by the equation, $I = \frac{D}{t^x}$, or even more complex forms of expression.

Fig. 6 shows a series of curves representing the ratios between the intensity of precipitation for any period and the precipitation for a period of 60 min. The full line represents the curve assumed by Mr. Buerger, $\frac{I}{K} = \frac{A}{20 + t}$. The other curves represent the ratios which would be obtained from rainfall curves proposed by several engineers for various localities. It is seen that, although Mr. Buerger's curve

* *Proceedings*, Am. Soc. C. E., for August, 1914.

follows, in general, near the median line, the variations are very considerable, so that, if the rainfall curves presented for the other localities are even approximately correct, material errors might be introduced by the use of the equation adopted by Mr. Buerger. Mr. Sherman.

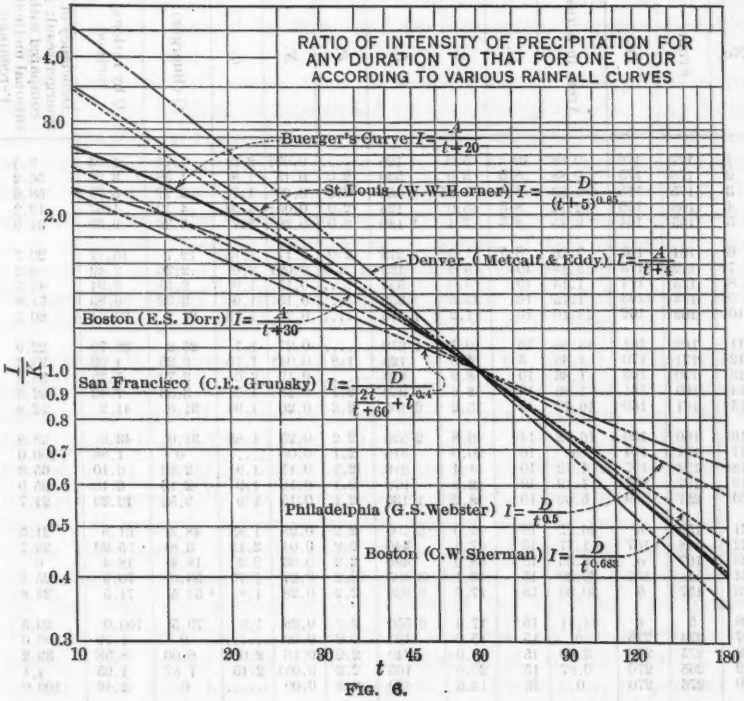

FIG. 6.

After all, however, the crucial test of the proposed method, as stated by Mr. Buerger, is the accuracy with which it reproduces the results to be obtained by the direct application of the rational method. The speaker has carried such a test farther than Mr. Buerger has done in his paper by applying it in detail to each step of the problem* presented by W. W. Horner,† Assoc. M. Am. Soc. C. E., with the results given in Table 16.

From Table 16 it is at once apparent that only in one case of the forty-two computations tabulated does the result obtained by Mr. Buerger's method agree with that computed by the rational method. In every other case Mr. Buerger's method gives a less result, and usually a considerably less result. In five cases the Buerger method

* As recomputed in detail for Metcalf and Eddy's "American Sewerage Practice."
† *Engineering News*, Sept. 29th, 1910.

Mr. Sherman.

TABLE 16.—COMPARISON OF RESULTS OBTAINED BY BUERGER'S FORMULA WITH THOSE OBTAINED BY THE RATIONAL METHOD, FOR A SEWER DISTRICT IN ST. LOUIS. C ASSUMED AS 0.50; $K = 1.61$.

No.	From.	To.	Area, in acres.	m.	s.	L.	P.	N.	q.	Q (Buerger).	Q by rational method.	Deficiency of Buerger result as compared with rational method—Percentage.
1	177	176	0.88	10	5.0	125	.....	0.09?	2.0	1.76	1.94	9.3
2	176	175	0.88	8.5	5.0	540	2.0	0.15	1.8	1.58	3.62	56.2
3	175	165	1.63	8.5	7.15	950	2.0	0.20	1.7	2.77	6.71	58.6
4	166	165	2.12	8.5	15.0	125	2.0	0.05?	1.94	4.10	4.67	12.2
5	165	164	3.75	8.5	7.4	1 180	2.0	0.20	1.7	6.39	9.39	31.9
6	164	163	7.26	8.5	11.5	1 275	2.0	0.17	1.75	12.7	15.97	20.7
7	172	173	1.13	10	5.0	125	.....	0.09?	2.0	2.26	2.49	9.2
8	173	174	1.13	10	5.0	550	.....	0.15	1.9	2.15	3.94	45.5
9	174	163	1.75	10	15.5	950	.....	0.15	1.9	3.32	6.89	51.8
10	163	162	12.10	10	11.2	1 475	.....	0.25	1.75	21.2	26.64	20.2
11	162	161	14.86	10	10.7	1 670	.....	0.27	1.7	25.2	32.70	22.9
12	171	170	1.86	3	15.0	120	1.8	0.04?	1.75	3.26	4.09	20.6
13	170	169	1.86	10	8.9	630	.....	0.12	2.0	3.72	5.75	35.3
14	169	161	1.86	10	12.6	940	2.1	0.18	1.9	3.53	7.40	52.3
15	161	160	16.72	12	15.2	2 105	2.2	0.20	1.9	31.8	41.2	22.8
16	160	159	16.72	14	16.8	2 520	2.2	0.25	1.85	31.0	43.6	28.8
17	131	194	0	10	20.0	310	2.1	0.00	.....	0	1.88	100.0
18	194	197	1.12	10	11.4	810	2.1	0.15	1.9	2.13	6.10	65.0
19	197	257	1.12	10	12.8	970	2.1	0.18	1.9	2.13	6.10	65.0
20	257	159	5.02	10	18.2	1 135	2.1	0.15	1·9	9.55	12.20	21.7
21	159	a	26.22	15	17.0	2 705	2.2	0.25	1.85	48.5	61.8	21.5
22	168	167	1.77	15	47.5	440	2.2	0.04	2.15	3.81	5.20	26.7
23	167	a	8.36	15	68.0	480	2.2	0.03	2.2	18.4	18.4	0
24	a	157	30.30	15	18.1	3 105	2.2	0.27	1.75	53.0	70.9	25.3
25	157	b	30.30	15	17.9	3 330	2.2	0.28	1.8	54.5	71.5	23.8
26	b	c	44.11	15	17.4	3 550	2.2	0.28	1.8	79.5	100.0	20.5
27	274	275	0	15	15.0	405	2.2	0.00	.....	0	1.19	100.0
28	275	270	3.00	15	12.0	840	2.2	0.13	2.0	6.00	8.98	33.2
29	265	270	0.87	15	20.0	165	2.2	0.05?	2.15	1.87	1.95	4.1
30	276	270	0	15	12.5	420	2.2	0.00	.....	0	2.46	100.0
31	270	269	12.02	15	18.2	965	2.2	0.08	2.1	25.3	28.9	12.4
32	269	c	13.98	15	20.0	1 005	2.2	0.09	2.05	28.6	31.6	9.5
33	c	155	46.89	15	17.2	3 675	2.2	0.27	1.75	82.0	104.0	21.2
34	271	272	0	15	25.0	400	2.2	0.00	.....	0	1.19	100.0
35	272	155	1.38	15	36.8	840	2.2	0.09	2.05	2.84	5.36	47.0
36	278	277	0	15	15.0	310	2.2	0.00	.....	0	1.81	100.0
37	277	155	2.39	15	18.3	460	2.2	0.08	2.1	5.03	5.85	14.0
38	155	d	52.29	15	16.9	3 825	2.2	0.28	1.8	94.2	117.0	19.5
39	d	154	54.86	15	16.6	4 000	2.2	0.31	1.7	93.4	122.0	23.4
40	154	e	54.86	15	16.4	4 140	2.2	0.32	1.7	93.4	122.6	23.8
41	e	153	57.58	15	16.0	4 440	2.2	0.33	1.7	98.2	126.0	22.0
42	153	136	59.21	15	15.7	4 600	2.2	0.34	1.7	100.6	128.6	21.8

shows a deficiency of 100%, or indicates that there would be no flow, whereas the rational method indicates an appreciable quantity of water to care for. These cases are at the upper ends of the sewers, where there is no drainage area tributary to the street inlets, but where allowance is made in the rational method for the probability

of roof-water drains being connected with the sewers. In such a case, the formula method necessarily fails, and it is necessary to apply judgment, that is to say, the rational method. Neglecting the cases where the deficiency as tabulated is 100%, the deficiency in the quantities computed by Mr. Buerger's, as compared with the rational method, ranges from 0 to 65%, and by inspection averages probably at least 25 per cent. Mr. Sherman.

It is possible, of course, that the speaker has misinterpreted some portion of the method recommended by Mr. Buerger, or he may have made mistakes in reading from the complex and small-scale diagrams accompanying the paper. He can only say that he has studied the paper carefully and endeavored to apply the method correctly, in accordance with the description, and to read the diagrams as closely as is ordinarily feasible in handling diagrams of this character. As a result of this test, the speaker is forced to the conclusion that Mr. Buerger has failed in his attempt to obtain a formula which will give results approximating those of the rational method.

ROBERT E. HORTON,* M. AM. SOC. C. E. by letter).—The writer agrees with Mr. Buerger that the McMath, Bürkli-Ziegler, and similar storm-water run-off formulas have been given practical application far beyond justifiable limits. On the other hand, he is not convinced, from anything which appears in the paper, that the Buerger formula is essentially an improvement, or that the Engineering Profession, if properly informed on the subject, does or would prefer to use a complicated formula such as the author presents, rather than the so-called rational method. This statement is based principally on the fact that the method of derivation of the formula, the specific data, or the assumptions on which it is based, are not given in the paper. The writer feels obliged to state that, in his opinion, the author is in duty bound, in fairness to the Profession, to present in full the derivation of his formula, if he expects it to be adopted generally. Mr. Horton.

In reference to this the writer feels justified in asking for an explanation of the necessity for the use of so complex and awkward a formula—a veritable *cheval de frise,* in fact. The formula contains some sixteen terms, and, if it was capable of direct solution, would require at least an equal number of mathematical operations, but it is not only an implicit formula incapable of direct solution, but even when the necessary factors (P and N) are given, it is only capable of solution by the use of a complicated diagram or by process of trial and error. The writer hesitates to accept the proposition that engineers would prefer to work out the run-off from a system of diagrams requiring such complicated operations as are necessary even

* Albany, N. Y.

Mr. Horton.

for the graphical solution of this formula, rather than use the rational run-off method in which direct calculations can be applied.

The formula, it appears, involves only six factors: the length of the sewer, the run-off ratio, the tributary area, the slope of the sewer, the slope of the ground surface, and the rainfall rate. After all, the most important elements governing the results are left to judgment, as in the case of the older formulas; and, furthermore, it is difficult to see just why so thorny and centipedal an expression is necessary to express the relations between these elements. In the hope that the formula is really as good and useful as its author apparently believes it to be, the writer feels amply justified in calling for full explanation of the assumptions on which it is based and the method of its derivation, and, furthermore, for the presentation of adequate reasons for the use of so complicated an expression.

The case is by no means parallel to that of the use of the Kutter formula. The latter is in reality a much simpler expression, although so complicated that its direct solution is seldom undertaken, but it did not profit by its complexity. As a matter of fact, it seems to have been well established that the original and simpler formula, derived without taking into account the Mississippi River experiments—later proved to have been erroneous—would have been far better; but, assuming, as is no doubt the case, that the only use of either formula would generally be through the medium of graphical solution, the Kutter formula can be solved by a single operation, with Roe's diagram, and by only two operations, with Kennison's diagram. The solution of the Buerger formula, on the other hand, requires ten operations on three diagrams. Undoubtedly, these diagrams, if on a large enough scale, could be read with accuracy, but their use seems to the writer to be very tedious and the result somewhat uncertain on account of the danger of accumulated error in reading from one intersection point to another, where there are several operations to be performed on the same diagram.

Mr. Grunsky.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—As the result of an interesting analysis of available run-off data, the author has worked out an approximation formula which may turn out to be better than the older formulas of a similar nature, such as those by Bürkli-Ziegler and McMath.

In this new formula, however, as in all others for the determination of the maximum run-off, the element of greatest uncertainty is the factor based on the relative imperviousness of the surface of the water-shed. This being the case, and all approximations of this factor being largely dependent on the judgment of the engineer, the question arises: Why should it seem necessary to substitute a complicated

*San Francisco, Cal.

formula for such simple expressions, having a rational and theoretic basis, as those presented by the writer in his paper on "The Sewer System of San Francisco and a Solution of the Storm-Water Flow Problem"?* Mr. Grunsky.

Using Mr. Buerger's notation,

$$q = \frac{4.36\,C\,K}{\frac{2\,t}{t+60} + t^{0.4}},$$

is the formula for the maximum run-off, in cubic feet per second per acre, if the San Francisco type of limiting rain intensity curve is adopted, and

$$q = \frac{5\,C\,K}{\sqrt{t}},$$

in case the parabolic form of limiting curve is preferred.

For comparison, results computed by these formulas, and containing the same elements as those used by the author, are presented in Table 17, which is the author's Table 13, with two new columns added.

TABLE 17.—COMPARISON OF VARIOUS FORMULAS FOR STORM-WATER FLOW.

All elements as assumed by the author for Table 13.

$S = 4$.

Area, in acres.	q = MAXIMUM RUN-OFF, IN CUBIC FEET PER SECOND PER ACRE.				
				Grunsky.	
	Bürkli-Ziegler.	McMath.	Buerger.	Parabola type, $C_1 = 7.75$.	San Francisco type, $C = 6.14$.
(1)	(2)	(3)	(4)	(5)	(6)
10	1.42	1.27	1.00	0.88	0.85
50	0.95	0.92	0.85	0.69	0.70
100	0.80	0.80	0.80	0.63	0.63
200	0.67	0.70	0.78	0.57	0.59
500	0.54	0.58	0.68	0.47	0.50
800	0.48	0.53	0.62	0.45	0.47
1 000	0.45	0.50	0.59	0.42	0.45
2 000	0.38	0.44	0.54	0.38	0.41
5 000	0.30	0.36	0.46	0.32	0.35
8 000	0.27	0.33	0.41	0.30	0.34
10 000	0.25	0.32	0.38	0.29	0.32

The critical time, t, in the absence of any information relating to the shape of the water-sheds, was determined from the length of the conduits as assumed by the author and from velocities of concentration, ranging from 3 ft. per sec. in the smallest areas to 6 ft.

* *Transactions*, Am. Soc. C. E., Vol. LXV, p. 294.

Mr. Grunsky,

per sec. in the largest ones, noted in the table. To this 5 min. were added for the time required to reach the sewer inlets. It was assumed that the critical time, t, would be about two-thirds of that required for water to flow to the gauging point from the most remote parts of each area. This method of approximating the critical time is not advocated as being a satisfactory one, but is here adopted merely to obtain relatively consistent results for comparison with those obtained by other formulas.

The successful application of the formulas suggested by the writer depends on two things: First, the determination of the limiting rain curve for the region under consideration; and second, the approximation of a value for t. Neither of these presents any serious difficulty, one being readily deduced from rain records and the other from the topographic features of the area for which maximum discharge is to be estimated. It is not believed that the type of formula for maximum run-off which these older formulas represent, can be materially improved.

The author accepts for rain intensity the formula:

$$I = \frac{80\,K}{20 + t},$$

this being of the type which has found acceptance by many engineers in the East.

The writer, on the other hand, has suggested

$$I = \frac{C}{\frac{2\,t}{t + 60} + t^{0.4}},$$

or

$$I = \frac{C_1}{\sqrt{t}},$$

where C and C_1 are constants to be determined for any locality from the rain records.

The eastern type of formula has the defect that it cannot be used for large values of t, etc. To illustrate this fact, the following comparison is made:

For $K = 1.00$ in., the eastern formula, as adopted by the author, would make

$$I = \frac{80}{20 + t}.$$

The writer's San Francisco formula would make

$$I = \frac{6.14}{\frac{2\,t}{t + 60} + t^{0.4}}.$$

Mr. Grunsky.

The writer's parabolic formula would make

$$I = \frac{7.75}{\sqrt{t}}.$$

The quantity of rain in t minutes, according to these several formulas, would be, for the special case of $K = 1.00$:

By the author's eastern formula..... $\dfrac{1.33\ t}{20 + t}$.

By the writer's San Francisco formula..... $\dfrac{t}{60}\left(\dfrac{6.14}{\dfrac{2\ t}{t + 60} + t^{0.4}}\right)$.

By the writer's parabolic formula..... $\dfrac{\sqrt{t}}{7.75}$.

TABLE 18.—COMPARISON OF VARIOUS FORMULAS FOR MAXIMUM RAIN INTENSITY.

The total quantity of rain in various time periods as determined by various formulas.

Time, in minutes.	Inches per Hour.			Total Quantity of Rain.		
	Buerger.	Grunsky.		Buerger.	Grunsky.	
		San Francisco.	Parabola		San Francisco.	Parabola.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
5	3.20	3.00	3.46	0.27	0.25	0.29
10	2.67	2.19	2.45	0.45	0.37	0.41
30	1.60	1.34	1.41	0.80	0.67	0.71
60	1.00	1.00	1.00	1.00	1.00	1.00
120	0.57	0.75	0.71	1.14	1.50	1.41
240	0.31	0.63	0.50	1.23	2.52	2.00
600	0.13	0.42	0.32	1.29	4.20	3.16
1 440	0.055	0.30	0.20	1.32	7.20	4.90

Table 18 gives a comparison of the rain intensities and quantities of rain, as determined by these expressions. It will be noticed that, although there is a sufficiently close agreement between them for time periods of less than 1 hour—at which point, on the assumption of $K = 1.00$, they are made to agree—the departure of the results by the eastern formula from those obtained by the formulas recommended by the writer becomes very marked when the value of t is materially greater than 60 min. The constants for the writer's formulas can be determined when the maximum quantity of rain for any time period up to 24 hours, or even longer, is known. This is not the case for the eastern type of intensity formula, the use of which, for this reason, should be discouraged.

Mr. Grunsky.

A mere inspection of Table 18 will show that the quantities of rain computed by the formulas, $I = \frac{80}{20+t}$ and $I = \frac{1.33\ t}{20+t}$, fall far short of what is known to be the fact. In any locality in which the rainfall in 1 hour is 1.00 in., the greatest possible rainfall in a day will certainly exceed 1.32 in.

Either of the two formulas for rain intensity suggested by the writer* is good for low values of t, and is also applicable to much longer periods of time than ordinarily come under consideration in urban or suburban run-off studies.

At Kansas City, Mo., during the night of September 6th and the morning of September 7th, 1914, the following quantities of rain fell:

Greatest quantity in	5	min	.....	0.64	in.
" " "	10	"	.....	1.01	"
" " "	15	"	.....	1.26	"
" " "	30	"	.....	1.56	"
" " "	1	hour	.....	1.97	"
" " "	2	hours	.....	2.78	"
" " "	5	"	.....	4.76	"
" " "	10	"	.....	6.94	"

Accepting for purposes of comparison the greatest recorded rainfall of this storm during 1 hour as the value of K, that is $K = 1.97$, then the additional comparison in Table 19 can be made, which again indicates the superiority of the writer's formula (of either the San Francisco or the parabola type) when the duration of the rain is materially greater than 1 hour.

TABLE 19.—COMPARISON OF THE RAINFALL AT KANSAS CITY, MO., WITH RESULTS BY FORMULAS.

Time, in minutes. (1)	For $K = 1.97$. Greatest Quantities of Rain, in Inches. Observed at Kansas City. (2)	Buerger. (3)	Grunsky, San Francisco $C = 12.1$. (4)	Grunsky, Parabola $C = 15.3$. (5)	Quantity of Rain. Grunsky, Parabola $C = 18.0$. (6)
5	0.64	0.53	0.50	0.57	0.67
10	1.01	0.88	0.72	0.80	0.95
15	1.26	1.13	0.90	0.98	1.16
30	1.56	1.58	1.32	1.39	1.64
60	1.97	1.97	1.97	1.97	2.33
120	2.78	2.26	2.98	2.78	3.29
300	4.76	2.46	5.28	4.40	5.20
600	6.94	2.54	8.23	6.22	7.35

* *Transactions*, Am. Soc. C. E., Vol. LXV, p. 422.

Mr. Grunsky.

It is probable that the actual limiting rain curve for Kansas City for low values of t will fall a little beyond either of the curves indicated by the figures in Columns 4 and 5. If the rainfall records of this September storm were to be made the guide in establishing a limiting curve for short time periods, the constants in the formulas, in other words, would be determined from the rainfall in 15 and 30 min. rather than from the fall in 60 min.

For a limiting rain intensity curve determined from such a single storm record as that in September, 1914, at Kansas City, the writer would recommend that C_1 be introduced into the parabolic formula at a value of 18. That is, for such a special case, there would be:

$$I = \frac{18}{\sqrt{t}}.$$

It may be added, in conclusion, that the simplicity of its form and close correspondence with the observed facts, both for short and for fairly long time periods, are elements in favor of the use of the parabolic type of limiting rain curve.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT.

Discussion.*

By Messrs. F. T. Robson and L. J. Le Conte.

Mr. Robson.

F. T. Robson,† Assoc. M. Am. Soc. C. E. (by letter).—The author has mentioned briefly the water supply—as developed and as proposed—of the East Bay shore. Several statements are startling, to say the least, and although the facts as given may be appreciated by engineers who have made a special study of the matter, yet to the Profession at large, and particularly to the local public, the acute water situation of the cities of Alameda and Contra Costa Counties comes as a severe shock. The public has long known of the unsatisfactory financial condition of these water companies, but has not been so well informed as to the water supply.

Water Supply of the East Shore Region of San Francisco Bay.

San Francisco has dominated everything in Central California for such a long time that the possibility that she will soon be passed in population and industrial growth by the cities on the East Shore has not come to be realized very generally. The cities of the East Bay region have been content to let their sister city make the fight for water alone, expecting to share in the spoils when the fight was won. It appears, however, that it will be too late to wait until San Francisco has brought in mountain water; by that time the growth of the East Shore will have been much retarded. The seriousness of such a situation deserves more emphasis and detailed analysis

* This discussion (of the paper by H. T. Cory, M. Am. Soc. C. E., published in September, 1914, *Proceedings*, and presented at the meeting of November 4th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† San Francisco, Cal.

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than that given by Mr. Cory in his paper, and it is felt that some additional facts may prove of interest.

In general, the present supply is a follows:

THE PEOPLES WATER COMPANY.

Historical.—As stated by Mr. Cory, the Peoples Water Company now supplying the greater part of the water used in the mainland cities, is the successor of nine lesser companies which have entered the field in the past forty-five years. Each supplied water to one or more of the transbay cities when they were much smaller than they are at present.

The consolidated plant does not constitute a well-designed and well-built water system. Although materials of an inferior grade were used extensively before consolidation, a big improvement has since been made. Of necessity, a number of features are obsolescent because of the rapid increase in population.

A very large part of the population lies below the 150-ft. contour, but the most desirable residence districts are on the foot-hill slopes to and beyond a 600-ft. elevation. Obviously, the water supply should be divided into at least three vertical zones with storage for each. In general, this has not been done, and could not be expected in a plant resulting from the consolidation of a number of disjointed systems, largely unrelated, and often duplicated through competition.

The main developments were as follows:

The Contra Costa Water Company was formed in 1866 to supply the small City of Oakland, and took water from Temescal Creek, later building the Temescal Reservoir.

Somewhat later the Sausal Water Company began supplying Brooklyn, a part of the present City of Oakland, with water from Sausal Creek.

The California Water Company began operations a little later to supply the Town of San Leandro, from San Leandro Creek, and began the construction of the San Leandro Dam. Both these companies were absorbed by the Contra Costa Water Company, which, in 1875, completed the San Leandro Reservoir (Lake Chabot).

The Alameda Artesian Water Company was formed to supply Alameda, then a small detached town. A group of pumped Artesian wells near Fitchburg was the source of supply. When the population reached 14 000 this became inadequate, and as the only additional supply then possible, because of cost, was the surplus water of the reorganized Contra Costa Water Company, the latter, in 1900, took over the Alameda Company.

The Piedmont Spring Water and Power Company—1892—furnished what is now Piedmont and part of Oakland with water from

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tunnels driven in the hills back of Piedmont, in Diamond Canyon, and another part of Oakland from wells in Oakland.

The Oakland Water Company, about 1893, was to supply a part of Oakland and the Town of Alvarado, by a pumping plant drawing on Artesian wells near Alvarado. Within a year it acquired the the Piedmont Spring Water and Power Company.

The East Shore Water Company was organized about 1898, by interests friendly to the Contra Costa Water Company, and soon after took over the Alameda Water Company. It supplied the higher parts of Berkeley and East Berkeley—then several miles north of Oakland—with water from Wildcat, Cordonices, and Wilson Creeks, from Summit and Berryman Tunnels, and from Pfeiffer Springs. When these sources proved inadequate, the only available increase within the limits of available capital was a surplus of the Contra Costa Water Company, and, therefore, the latter company soon took over the East Shore Water Company.

Prior to the advent of the Piedmont Spring Water and Power Company, in 1892, the Contra Costa Water Company, by acquiring the Sausal and California Water Companies, was in exclusive possession of the field. In November, 1894, the Oakland Water Company (which had acquired the Piedmont Spring and Water Company) first supplied Oakland with water. For the next five years competition was severe and wasteful—the Oakland Water Company paralleling more than 100 miles of the Contra Costa Water Company's mains in Oakland, the only territory invaded. In 1899 these competing companies consolidated, the Contra Costa Water Company continuing the business.

Thus, by the end of 1899, the Contra Costa Water Company had, willingly and unwillingly, acquired and merged seven water companies.

The Pinole Water Company was formed in 1905 to take water from the Pinole Creek water-shed and supply a part of the Contra Costa Water Company's territory, the surplus of the latter Company's supply having been exhausted by the greatly increased demand. In a few months, it was purchased and merged—before any works were completed.

In 1906 the Syndicate Water Company was formed by the financial interests behind the Realty Syndicate, which, in 1904, started acquiring and developing on an immense scale the wonderfully attractive foot-hill suburban property on the East Bay side. One part of its programme resulted in the building of the Key Route Ferry and suburban electric railway system, now carrying more than 14 000 000 passengers annually. The deficient and unsatisfactory water supply of Oakland and Berkeley threatened the success of the Syndicate's plans. Rights and holdings were acquired on San Pablo and San Leandro Creeks, and construction was begun to develop enough water

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to afford—with that of the Contra Costa Water Company—a supply for several years. After some negotiations, the new interests took over the old company and merged under the name of the Peoples Water Company on January 1st, 1907. At that time there were 28 000 connections, mostly unmetered, and 18 000 000 gal. per day were supplied. At present, this corporation has about 60 400 taps, 99% of which are metered; yet the consumption has been kept at about the same quantity as it was 7 years ago. Formerly, a wholesale rate was made to industrial plants and large consumers, but, when the consumption had almost reached the limit of the supply, wholesale rates were discontinued, with the result that many private wells were sunk where there was the slightest chance of securing water. The high price of water has had a bad effect on industrial growth.

Essential Engineering Features.—Fifteen sources of water supply, comprising four surface-water and eleven ground-water sources, have been used during the past 10 years by the Peoples Water Company and its predecessors, as shown on the map, Fig. 24. For the use of this map, and also the profile, Fig. 25, containing data regarding these sources of supply, the writer is indebted to the courtesy of J. H. Dockweiler, M. Am. Soc. C. E.

Lake Chabot, also called San Leandro Lake or San Leandro Reservoir, with a water-shed of 41 sq. miles, was created by an earthen dam, 125 ft. high, across San Leandro Creek. The elevation of the top of the dam is 242 ft. above sea level, and, as reduced by silting, the reservoir now has a capacity of 4 840 000 000 gal., or 14 858 acre-ft. Water flows from the reservoir to a battery of Hyatt mechanical filters, then to a clean-water basin, and thence by gravity to the 38½-in. riveted steel main leading to Oakland and connecting on East 14th Street, near High Street, with the main from the Alvarado and Fitchburg wells. There, the three waters are mingled and subsequently picked up at the 24th Avenue pumping station for distribution. The dam built in 1875 was raised 10 ft. in 1892, and the flow line now contains 447 acres. In the past 10 years, the draft has varied from a maximum of 3 698 000 000 gal. during 1907, to a minimum of 2 152 000 000 gal. during 1911. The safe and dependable draft is presumed to be 6 000 000 gal. per day.

Temescal Reservoir, about 4½ miles northwest of the Oakland City Hall, has a capacity of 150 000 000 gal., or 460 acre-ft., and the flow line includes 16.75 acres. It is created by an earthen dam, 40 ft. high, across Temescal Creek, has 3½ sq. miles of water-shed, and a flow-line elevation of 436 ft. Water is carried by a 16-in. riveted steel pipe through five mechanical filters (one Hyatt and four Loomis) to a clean-water reservoir, with a capacity of 750 000 gal., at an elevation of 375 ft. Thence it is sent to Berkeley and Oakland by gravity. The draft from this source varies from a maximum of

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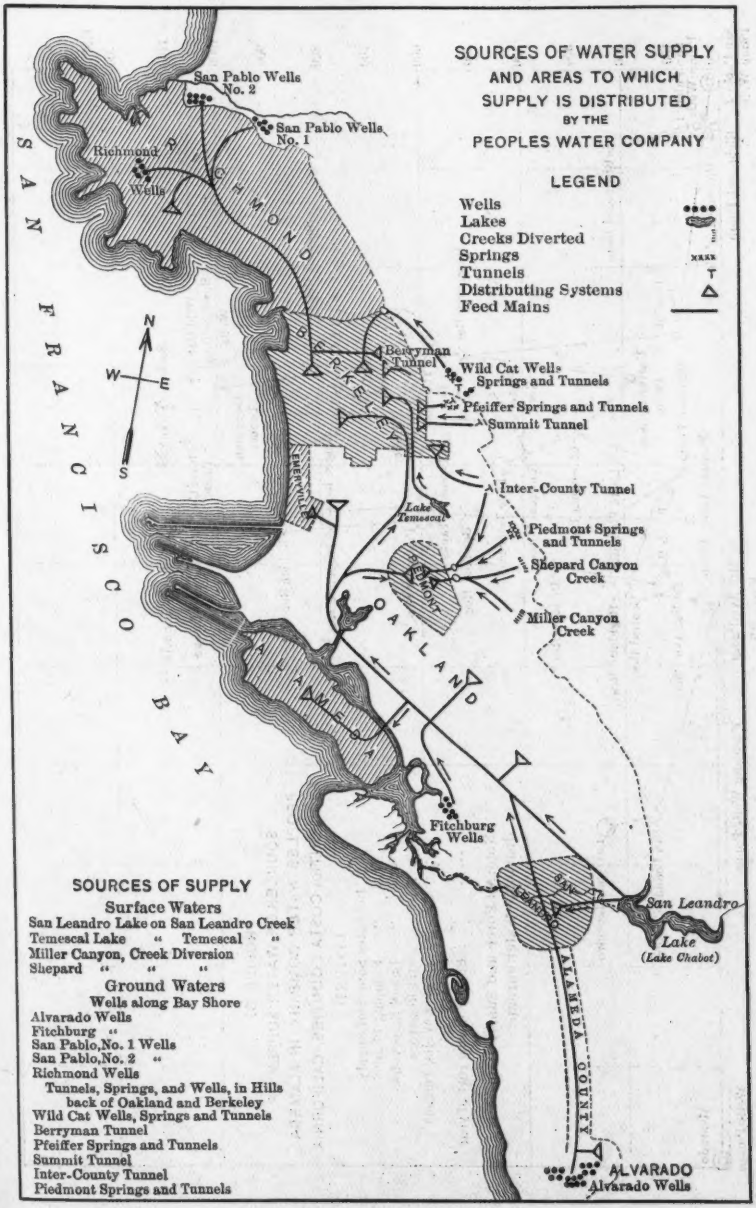

FIG. 24.

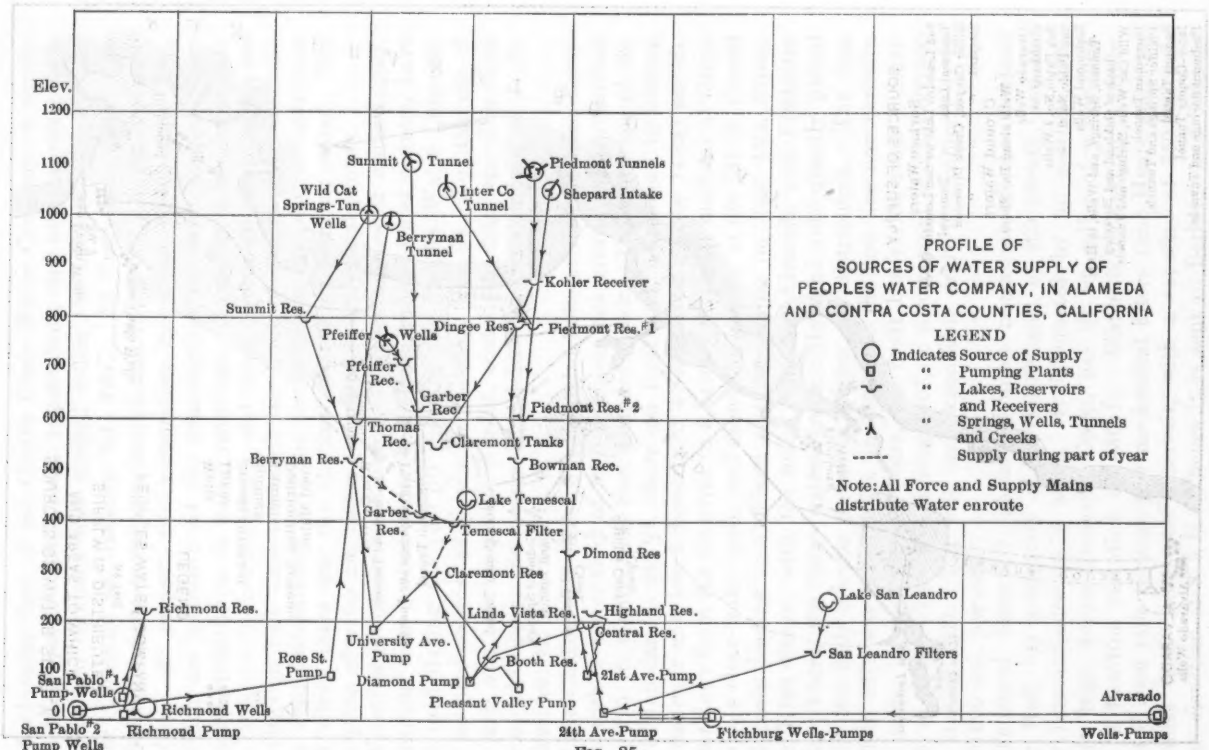

FIG. 25.

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206 500 000 gal., in 1906, to a minimum of 126 000 000 gal. in 1904. The safe and dependable draft is said to be 400 000 gal. per day. Mr. Robson.

Shephard Canyon waters are diverted from the north fork of Sausal Creek, in Shephard Canyon, by a small wooden dam, and through a 7-in. riveted steel pipe leading into Piedmont Reservoir No. 1. Thence it is conducted to Piedmont Reservoirs Nos. 2 and 3, and supplies the higher levels of Piedmont. The draft has varied from a maximum of 32 000 000 gal., in 1907, to 25 000 000 gal., in 1910.

The Miller Creek Canyon water is diverted from the east fork of Sausal Creek, and conveyed by a 2-in. pipe to connect with a 7-in. pipe leading from the Shephard Canyon source to Piedmont Reservoir No. 1.

The ground-water supplies consist of about 150 wells strung along the east shore of the Bay in five localities, from Alvarado to north of Richmond. At present these supplies are being drawn on for about 12 000 000 gal. per day, of which 85% comes from about 35 wells near Alvarado. From tunnels, wells, and springs in the hills back of Oakland and Berkeley—five sets—about one-tenth as much water is obtained as from underground sources along the Bay Shore, say, 1 000 000 gal. per day, of which about one-half is from Wildcat wells and tunnels. These ground-water supplies are not filtered.

In addition to the four impounding reservoirs, Lake Chabot, Temescal, and Piedmont Nos. 1 and 2 (which have a combined capacity of 5 011 000 000 gal., or 15 384 acre-ft.), there are thirteen distributing reservoirs with capacities of from 158 000 000 to 28 000 gal. (total 240 000 000 gal.), and at elevations of from 80 to 950 ft.

There are five primary and six "booster" pumping stations, with capacities of 16 500 000 and 35 600 000 gal. per day, respectively, with 10 000 000 and 20 000 000 gal. as a daily average. Of the total pumpage about 48% is pumped once, 33% twice, 18% three times, and 1½% four times.

As has been said, the distribution system has about 800 miles of pipe, of which about 48% is 2-in. and less, and 50% 4-in. and less; 35% is cast iron, from 3 to 16-in.; 10% iron and steel, riveted, from 4 to 37-in.; 50% standard screw, from ⅛-in. to 8-in.; and the remainder, converse, cement-lined casing and tubing.

Union Water Company of California.

The Union Water Company is the next in importance serving those communities directly across the Bay from San Francisco. It resulted from the activities of the Bay Cities Water Company formed in 1902 to supply water to all the Bay cities.

Historical.—When, in 1901, C. E. Grunsky, M. Am. Soc. C. E., then City Engineer of San Francisco, first reported on a Sierra water supply of 60 000 000 gal. per day, many engineers believed such a

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quantity of water could be obtained more cheaply from near-by sources. This fact and the friction between the Spring Valley Water Company and San Francisco on the peninsula and the Contra Costa Water Company and the East Bay cities on the mainland, were the underlying reasons for the formation of the Bay Cities Water Company, on October 21st, 1902. The Company acquired rights and large land holdings and outlined a project consisting of two parts—the Mt. Hamilton Project and the Coyote Project. The former is said to be capable of being developed to supply 22 000 000 gal. per day and the latter 40 000 000 gal. per day, or, combined, 62 000 000 gal. per day, approximately the quantity Mr. Grunsky planned to bring in by the initial Tuolumne, better known as the Hetch Hetchy, supply.

The Bay Cities Water Company made offers to both San Francisco and Oakland, but both were declined. Suits were instituted by the Spring Valley Water Company attacking the right to divert water from the Mt. Hamilton Project which is higher up but on the same Alameda Creek drainage area as that Company's Calaveras Reservoir and Sunol filter galleries, and by the land and riparian rights owners in the Santa Clara Valley, on the Coyote Project.

The Coyote Project, as proposed, consisted in collecting and storing surface waters in reservoirs to be constructed, one on Coyote Creek, another on Uvas Creek, and a third on Llagas Creek, all draining into the Santa Clara Valley. The decision of the Court in these suits will doubtless make it impossible to use this water in Alameda County.

The proposed Mt. Hamilton development would supply only 22 000 000 gal. per day, and therefore, even though the conflict with the interests of the Spring Valley Water Company did not exist, it would not be attractive, even for the East Bay communities alone. As a matter of fact, the water in question can be conserved more cheaply and delivered to any part of Greater San Francisco by the Calaveras Reservoir of the Spring Valley Company, because the latter is much nearer the mouth of Alameda Creek and the Bay cities.

In the course of these activities, the possibilities of the local underground waters were critically examined. H. L. Haehl, Assoc. M. Am. Soc. C. E., working in conjunction with C. E. Gilman, Assoc. M. Am. Soc. C. E., Manager, Edwin Duryea, Jr., M. Am. Soc. C. E., Chief Engineer of the Bay Cities and the Union Water Companies, and John W. Alvord, M. Am. Soc. C. E., Consulting Engineer, made exhaustive geological surveys, and reached practically the same conclusions, namely:

(1) That there are two distinct and disconnected bodies of underground water within the San Francisco Bay region, one reached by shallow wells less than 200 ft. deep, chiefly supplied by waters absorbed from stream overflow and the rainfall on the local alluvial

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fans, and to a much less, though considerable, extent by percolations through the joints and fissures of the rock rim and basin of the valley; and the other, locally known as the "Artesian" water, lying much deeper and in large measure supplied by the uptilted gravels of the Pliocene age, exposed at high elevations in the hills on the western side of the valley, where the rainfall is a maximum.

(2) That at the southern end the deep waters are truly Artesian, but the strength of the flow gradually diminishes toward the north, until in the region immediately south of Oakland the deep-seated strata no longer force their waters to the surface.

(3) That analyses of the waters of the two types taken from adjoining wells in general show no similarity, and the seasonal fluctuation of the respective water-tables are essentially different in time.*

(4) That local well-borers call these two underground water bodies "surface" waters and "Artesian" waters, respectively, and cast off the former in the region between San José and the southern extremity of the Bay, because they must be pumped.

(5) That the water supplied by percolations through the fault zones, joints, and fissures, in the distorted sedimentations of the rock rim and basin of the valley, although not the most important, is a considerable and material source of supply, and cannot be neglected in any proper consideration of the sub-surface water supply.

(6) That draft on the deep-seated "Artesian" waters west of the Alameda or Niles and San Lorenzo fans or "cones", is less than on the "surface" water, and that the former are not dependent for their supply on the surface flow of Alameda and San Lorenzo Creeks.

The Union Water Company, formed on December 21st, 1910, has put down a great many wells, scattered over the East Bay region from Richmond to Newark, and constructed several small distributing systems in various parts of the region. The Bay Cities Water Company is yet inoperative, and doubtless will never do any business in Greater San Francisco.

Essential Engineering Features.—The collection works of the Company, shown on Fig. 26, consist of 13 small pumping plants, of which 12 are now in use. In general, each well is an individual pumping plant with a gasoline engine or electrically-driven pump. The Company states that it has a total of 122 wells, in 14 groups, with a 24-hour test capacity of 26 000 000 gal. per day, and a safe daily draft of 19 000 000 gal. Of these wells, 12 are on the San Pablo cone, 65 on

* An interesting account of the geology and geological history of the region may be found in the Report of the Committee on the Geology of the Earthquake, in the Report on the Effects of the San Francisco Earthquake, *Transactions*, Am. Soc. C. E., Vol. LIX, 1907, pp. 216 *et seq.*, and in the Santa Cruz Folio, Geological Atlas of the United States Geological Survey, Washington. Reference may also be made to the discussion on the paper "Rain and Run-off near San Francisco, California," by C. E. Grunsky, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI, 1908, pp. 522 *et seq.*

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the San Leandro cone, 5 on the San Lorenzo cone, 14 on the Niles cone, and the remainder outside these cones. Owing to contracts for watering lands north of Newark, the output from Plants 11 to 14, inclusive (about 11 000 000 gal. per day), are practically not available for use elsewhere. Of the remaining 8 000 000 gal. per day (which it is said may be used in supplying present urban communities) 2 666 666 gal. per day come from 23 wells, Plant 5, and 3 333 333 gal. per day from 33 wells, Plant 6.

The distribution works consist of about 91.5 miles of pipe, forming six different systems, as follows:

Three small systems in Richmond with a total of 10.4 miles in connection with Plants 1, 2, and 3; 2.9 miles connected with Plant 4; 52.6 miles as a network connected with Plants 5, 6, 8, and 9; and 25.1 miles connected with Plant 10; the last two systems are tied together. Almost all the mileage is supplying the south end of Oakland and San Leandro. The pumps deliver directly into the mains the pressure being controlled by automatic electric devices.

Of the pipe 39% is 2-in. and less, 72% 4-in. and less, 95.6% 8 in. and less, and the remainder is 10 and 12-in. Cast-iron pipe, from 4 to 10-in., comprises 24%, and the remainder is converse lock-joint and steel. There are 3 100 taps, of which 92% are metered, and 850 000 gal. per day are being supplied at a base rate of 32 cents per 1 000 gal. In addition, some hydrants have been put in at an annual rental of $35 per annum.

Other Water Supply Systems.

Besides the two systems just described, there are six public utilities controlled by and reporting to the State Railroad Commission supplying potable water in the East Bay region of Greater San Francisco, and three municipally owned—by the Towns of Newark, Niles, and Pleasanton—or nine systems altogether. These secure ground-water from wells, except Newark and Niles, which buy water at wholesale from the Spring Valley Water Company.

TABLE 5.

Company.	Place.	Assets, latest balance sheet.	Base rate per 1 000 gal.
Bay Point Light and Water Company	Bay Point	$25 000	0.30
Black Diamond Water Company	Pittsburg	67 931	0.20
Citizens Water Company	Niles	5 000	Flat rate.
Hercules Water Company	Hercules and Pinole	177 457	0.50
Port Costa Water Company	Port Costa, Concord, Martinez	756 971	0.30
San Lorenzo Water Company	Haywards	52 200	0.40

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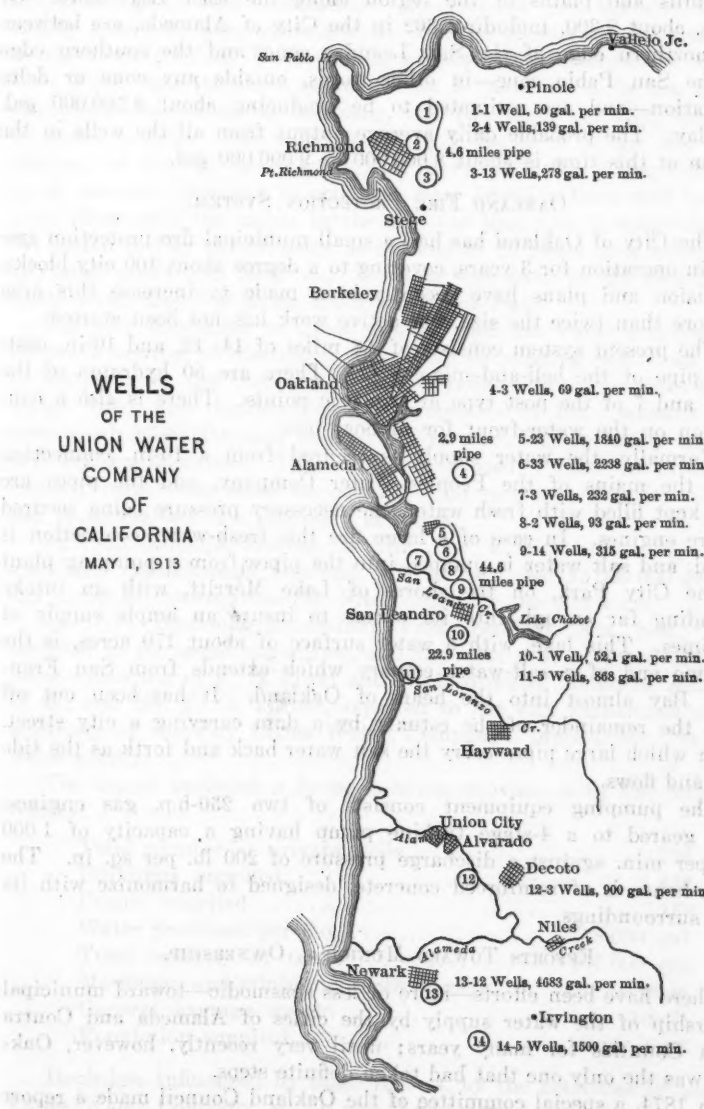

FIG. 26

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In addition, there are about 5 000 private wells in the western foot-hills and plains of the region along the East Bay shore. Of these, about 2 300, including 362 in the City of Alameda, are between the northern edge of the San Leandro cone, and the southern edge of the San Pablo cone—in other words, outside any cone or delta formation—and are estimated to be producing about 4 500 000 gal. per day. The probable daily average output from all the wells in the region at this time is about 8 000 000 or 9 000 000 gal.

Oakland Fire Protection System.

The City of Oakland has had a small municipal fire protection system in operation for 3 years, covering to a degree about 100 city blocks. Provision and plans have recently been made to increase this area to more than twice the size, but active work has not been started.

The present system consists of 3.8 miles of 14, 12, and 10-in. cast-iron pipe of the bell-and-spigot type. There are 50 hydrants of the flush and 7 of the post type at strategic points. There is also a connection on the water-front for fireboat use.

Normally, the water supply is secured from a 16-in. connection with the mains of the Peoples Water Company, and the pipes are thus kept filled with fresh water, the necessary pressure being secured by fire engines. In case of a large fire this fresh-water connection is closed, and salt water is pumped into the pipes from a pumping plant in the City Park, on the shores of Lake Merritt, with an intake extending far enough into its waters to insure an ample supply at all times. This lake, with a water surface of about 170 acres, is the extreme end of a salt-water estuary which extends from San Francisco Bay almost into the heart of Oakland. It has been cut off from the remainder of the estuary by a dam carrying a city street, under which large pipes carry the salt water back and forth as the tide ebbs and flows.

The pumping equipment consists of two 250-h.p. gas engines, each geared to a 4-stage turbine pump having a capacity of 1 000 gal. per min. against a discharge pressure of 200 lb. per sq. in. The pump-house is of reinforced concrete, designed to harmonize with its park surroundings.

Efforts Toward Municipal Ownership.

There have been efforts—more or less spasmodic—toward municipal ownership of the water supply by the cities of Alameda and Contra Costa Counties for many years; until very recently, however, Oakland was the only one that had taken definite steps.

In 1874, a special committee of the Oakland Council made a report on the local sources of water supply. It is interesting to note that, even as early as this, the merit of the Calaveras Creek dam site and

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water-shed was recognized clearly. The properties of the Contra Costa Water-Works were also investigated and found to promise an adequate supply. The committee recommended that the city immediately secure its own water system. Shortly thereafter, the president of the Contra Costa Water-Works proposed two alternatives to the city: First, that its plant be bought on a valuation to be set by five appraisers—two from the company, and two from the city, the four to choose a fifth. The value found by a majority of the five to be binding; or, second, that the city lend its credit to the Water Company to an amount sufficient to complete San Leandro Dam and connect it by pipes with the mains in the city—at that time 8 miles distant. An option to buy the property at the end of 10 years at a value fixed as stated would also be given. Nothing was done by the city in the matter, and thus Oakland's opportunity was lost.

The city continued to grow rapidly, and the water problem was always a source of discussion, but little or nothing was done until in March, 1900, the Company began its famous suit over rates before Judge Hart who decided that the properties of the Company were then worth $7 000 000 and allowed 7% interest, 2% operating expense, and 1% maintenance. This allowance caused a raise in rates, and many private wells were the result. The suit cost the city $32 000 and doubtless cost the Company as much.

In 1902, Rudolph Hering, M. Am. Soc. C. E., made a report for a citizens' committee of Oakland. He advised that 15 000 000 gal. per day could be drawn from the Niles cones for the city's use.

In 1903, Desmond FitzGerald, Past-President, Am. Soc. C. E., reported to the same committee, and the next year a Board of Engineers consisting of A. M. Hunt and J. M. Howells, Members, Am. Soc. C. E., and Mr. F. C. Turner, reported to the city authorities on the adequacy of the Bay Cities Water Company's project for Oakland's water supply.

The report included a house-to-house canvass, with the following results:

Total number of private wells	718
Buildings supplied	1 436
People supplied	7 180
Water produced per day	718 000 gal.
Total capacity of tanks	3 467 000 gal.
Maximum and minimum depth of wells	681 to 18 ft.
General average depth	100 ft.
Population supplied	10 per cent.

Doubtless influenced by these reports, by the activities of the Bay Cities Water Company, which was threatening to invade its territory, and by the controversy with the city over rates, the Contra Costa

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Water Company offered either to sell or lease to the city, at a value determined by two men who should select a third if necessary. The city took no steps, and the offer was finally withdrawn when rates were fixed on a valuation of $4 700 000, the city ignoring the decision of Judge Hart giving a value of $7 000 000. This case was then on appeal to the Supreme Court.

In June, 1904, Judge Gilbert granted an injunction against the rates fixed by the city, but required the Company to file a bond for $130 000.

In October, 1904, five water supply propositions were submitted to the city, chiefly by the Bay Cities Water Company which was making every endeavor to break into the territory; and in March, 1905, an election was held on an offer submitted by the Bay Cities Company. The campaign will long be remembered as the hottest and most spectacular known in Oakland. A large sum of money is said to have been spent by the opposing water companies. The vote resulted in 4 369 for and 5 207 against the proposition.

Since that time, the water problem has been before the people continuously. The suit started in 1904 has not yet been decided. The Contra Costa Water Company has merged with other companies, until now the corporation is known as the "Peoples Water Company". The Bay Cities Water Company has surrendered its activities to the Union Water Company.

The population has grown steadily, and the several communities on the East Shore now have the same water problem confronting them that Oakland left unsolved in 1874.

Financial and physical conditions make it almost impossible for one community to secure an adequate water supply without joining with others. This has been long realized, but the laws of California made no adequate provision for help. In 1909, a law following more or less closely the one under which the Metropolitan Water and Sewerage Districts of Boston and New York are operating, was passed by the State Legislature, but nothing has been done to organize a district because of alleged defects. At the request of the smaller communities, therefore, it was modified in 1911 to give them more representation. The present law permits any number of municipalities within the same county, as well as unincorporated territory, to unite and form a Municipal Water District.

In 1912, petitions were prepared and circulated to form a district, under the law of 1911, consisting of the municipalities of San Leandro, Alameda, Oakland, Emeryville, Piedmont, Albany, and some unincorporated territory, all in Alameda County. Sufficient signatures were finally secured, and it was thought a district would be formed. However, the Supervisors of Alameda County, who had the power to call the election, kept postponing the matter for one reason and another,

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until the present year, when an election was called for June. The Peoples Water Company put the stock control of the company, together with an option to sell its entire properties, the price to be fixed by the State Railroad Commission, in the hands of an Oakland bank.

Strong opposition to forming the district immediately appeared, a campaign of vituperation by the opponents followed, and attacks were made on the law, the water company, and the supporters of the measure. It seemed impossible to convince the electorate of the gravity of the situation, and even the majority of the newspapers were only lukewarm or actively opposed.

Due to the strenuous efforts of Mayor Heywood, of Berkeley, a majority of some 300 or 400 was given in favor of the district there, but this only nullified the adverse vote in the smaller communities, as Oakland returned a majority against the district of about 2 500. Less than 50% of the registered vote was cast. This defeat was a body blow to those who knew the conditions. Nothing has been done since, and it is not known what steps can or will be taken. A new law has been prepared, but should it be passed there will doubtless be as much opposition as before.

In the meantime, the Peoples Water Company was becoming more involved, financially, and in July, 1914, interest on its bonded indebtedness was defaulted. The affairs of the Company have been put in the hands of a committee of bankers, and protective committees of bondholders have been formed. No satisfactory reorganization plans have been given out, and the general financial conditions resulting from the war in Europe leave little of an encouraging nature apparent in the immediate future.

Conclusion.

No satisfactory solution of the water supply problem for the East Shore cities can be reached until these can be joined in one district or organization for that purpose. When this has been accomplished, such properties of the Peoples Water Company as are necessary must be purchased and the development of local sources completed. The district would then be in a position to purchase and distribute water from any source available.

Half of the total present and proposed supply of the Spring Valley Water Company comes from Alameda County sources. Under existing laws, if San Francisco purchases the Spring Valley Company, the taxes now levied by Alameda County on this property would not have to be paid by San Francisco. This would mean a large decrease in revenue for general county purposes.

After forming a district and purchasing the properties of the Peoples Water Company, the East Shore cities should buy the Ala-

Mr. Robson.

meda County system of the Spring Valley Company, thus relieving San Francisco of this portion of the purchase cost and making available a larger sum for acquiring distant mountain supplies.

It is a striking commentary on the foresight of the Spring Valley Company, when it is known that for years it has had a deeded right of way from its Calaveras Dam to the City of Oakland.

Mr. Le Conte.

L. J. LE CONTE,* M. AM. SOC. C. E. (by letter).—The writer is greatly interested in this timely paper. The author has certainly taken a very wide grasp of the whole complex situation, and in arriving at his conclusions has given due weight to all the important factors which influence a wise decision.

In comparing several schemes in which the construction cost is nearly the same, all the minor items—cost of operation, maintenance, depreciation, renewals, and incidental incomes—of course, are factors; but, when the first cost of construction is vastly greater in one case than in another, consideration of these smaller items can be omitted as they have no bearing whatever on the conclusion drawn.

Undoubtedly, the wisest policy for San Francisco is to develop, to the fullest extent possible, local supplies which will probably amount to 200 000 000 gal. per day, and be sufficient for 80 years to come. The city should then develop the pumping works at Moore's Landing on Old River, San Joaquin, and force the water up to Altamont Reservoir —capacity 1 000 000 000 gal. Thence supply mains should lead to the extensive natural gravel filter beds in Livermore Valley, and thence to the several large storage reservoirs in the Alameda Creek water-shed.

The first development will be for 50 000 000 gal. per day, which will be sufficient for 25 years more, or, say, until 2000 A. D. The cost of these new developments up to this time will be:

Peninsula	$40 000 000
Pumping works	10 000 000
Total	$50 000 000

thus making a unit cost for development of $200 000 per million gallons per day. Moreover, distant future expansions of the supply can be readily made at the same unit cost for development.

The East Bay region, by full development of all local sources, can obtain from 30 000 000 to 40 000 000 gal. per day, which will be sufficient for from 10 to 15 years to come. At the close of this period—1930—the best available outside source will be, likewise, at Old River, just above the junction of Indian Slough, some 12 miles above Marsh's Landing, which will be the site of the pumping works.

Thence, two supply mains will lead along the straits and bay shore to Berkeley, Oakland, and Alameda. The first development should

* Berkeley, Cal.

Mr. LeConte.

be for 100 000 000 gal. per day, which would call for the expenditure of $20 000 000. This additional supply would be sufficient for 60 years to come, or until 1975, and would show a unit cost of development of $225 800 per million gallons per day, delivered; and, moreover, the expansion of the supply, in the more distant future, can be readily made at the unit cost of $200 000 per million gallons per day.

These figures show conclusively that the distant Sierra supplies are undesirable, from every point of view, as will now be demonstrated.

In 1901, Mr. Grunsky reported that the Hetch Hetchy scheme, developed to 60 000 000 gal. per day, would cost $32 000 000, or $533 000 per million gallons per day, or nearly three times the cost of development of the local supplies near San Francisco. Likewise, in 1912, Mr. Freeman reported, with reference to the Hetch Hetchy scheme, that the first development for 160 000 000 gal. per day would cost (revised estimate) $64 000 000, showing a unit cost of development of fully $400 000 per million gallons per day. It would finally appear that this distant source of supply is vastly more expensive and is, therefore, undesirable.

Delta Sources.—The combined low-water flow of all the rivers naturally emptying into Carquinez Straits is now approximately 7 200 cu. ft. per sec., during the dryest seasons on record. This is simply the surface or visible flow, and does not include the underground supply which would probably increase it materially, if it were known; but, even the minimum (taken at 7 200), will give a reliable supply of 4 615 000 000 gal. per day. Now, it is a well-established fact that the large future hydraulic developments in the great Central Valley of California—in the way of irrigation, power, flood control, and extensive reclamation and irrigation works at and near Tulare Lake—will collectively tend to increase materially the low-water flow of all the rivers in the delta region.

The great works at Tulare Lake and vicinity will certainly add practically 10 000 sq. miles to the direct water-shed of the San Joaquin proper, and will certainly add 2 000 cu. ft. per sec. to the low-water flow of the river at a point just above Firebaughs. This result, combined with the underground return flow from the great irrigation fields on both sides of the river, will ultimately swell the low-water flow of the San Joaquin River to fully 6 000 cu. ft. per sec., thus greatly improving the low-water steamboat navigation on the entire river, to the everlasting benefit of the public in general.

The same beneficent results will also take place on the Sacramento River, and the writer is of the opinion, therefore, that fully 10 000 000 000 gal. per day can be considered as the reliable delta supply in the distant future.

Of course, subsidence and filtration, sooner or later, are to be expected for all supplies taken from the delta; but, naturally, this

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work is reduced to a minimum by placing the intakes on the south or Old River side, where long experience shows turbidity to be a minimum.

The quantity proposed to be taken from the delta supply, in the near future, is only 150 000 000 gal. per day, or 3% of the total, hence it can safely be said that there will be no trouble whatever from salt water.

In conclusion, it would appear that San Francisco and the East Bay region can both safely lay aside the idea of calling on the distant Sierra supplies for the next 100 years; and they will certainly be far better off financially by doing so.

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PAPERS AND DISCUSSIONS

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EXTERNAL CORROSION OF CAST-IRON PIPE.

Discussion.*

BY MESSRS. KENNETH ALLEN, WILLIAM W. BRUSH, SAMUEL TOBIAS WAGNER, R. C. KELLOGG, A. D. FLINN, WILLIAM J. BOUCHER, AND GEORGE M. PURVER.

Mr. Allen.

KENNETH ALLEN,† M. AM. SOC. C. E.—With respect to the Atlantic City pipe, there are two old cast-iron mains, one 6-in., laid in 1881, and one 20-in. laid about 6 years later. Between 1902 and 1905, when the speaker was there, these pipes had deteriorated greatly, as described by Mr. Pugh. Soft spots in the metal were frequent, and although the pipes were in service and held the pressure, it was felt that it was not good policy to depend on them. In places the metal could be whittled like chalk, and yet no failure occurred in the 6-in. pipe when it was tested with a pressure of 75 lb. per sq. in.

With regard to the 40-in. steel main, which was laid in 1901 beside the cast-iron mains: that pipe, which was constructed of ¼-in. open-hearth plates, corroded much more rapidly. Two small perforations were discovered on February 2d, 1905; fourteen by September 1st, and twenty-two by January 1st, 1906. On September 1st, 1905, the speaker estimated that it would cost $345 340 to duplicate the three mains, and that their actual value then was $187 100, showing a deterioration of 46 per cent.

An inspector with a kit of tools walked the length of the mains from the pumping station and back—about 12 miles—every day; and when he found a hole in the steel main, he would whittle a small stick and plug it up—a very simple and effective expedient.

It seemed to the speaker at the time that, in view of the cost of the pipe—about $200 000—it would be desirable to preserve it, if possible,

* Continued from October, 1914, *Proceedings*.

† New York City.

Mr. Allen.

by a casing of 3½-in. reinforced concrete. This, it was estimated, would cost about $58 000, or $11 500 per mile, where exposed to the meadow conditions. Before deciding on this, however, it was concluded to make a test by covering in this way several short sections containing leaks, a contract for which was let before the speaker left. Later, however, it was decided to abandon this main altogether, and to put in a 42-in. wood-stave pipe. Still more recently, the 48-in. cast-iron pipe, which Mr. Pugh has referred to, has been constructed. Cast-iron has always been very difficult to handle on the meadows and, on account of vibrations from railway trains near-by, difficult to maintain in a tight condition; but it was believed that it would be much more durable than steel and, if raised above the surface of the meadows on cradles, would be more satisfactory on the whole than wood-stave pipe.

The author states that the "soil was free from organic acids". It is understood that analyses made by W. P. Mason, M. Am. Soc. C. E., some years ago, indicated the presence of vegetable acids which, of course, would promote corrosion of the iron. This action may have been increased at certain points by electrolysis, as tests, made in July, 1905, indicated a fairly constant flow of 30 amperes through the two cast-iron and the 30-in. steel mains toward the mainland where the current passed off to street railway tracks and a local water main. Whether the alteration in the metal referred to by the author may have been due partly to this cause is not known, but is suggested as possible.

The winter of 1904-05 was one of great severity, and the 12-in. main froze for the first time, breaking twenty-two lengths of pipe. It seems to the speaker quite probable that the loss would have been less if the pipe had been new and sound.

In the Annual Report of the Water Department dated August 1st, 1901, W. C. Hawley, M. Am. Soc. C. E., then Engineer and Superintendent, stated that:

"The cast-iron of the pipe has been changed in many places to a condition such that it can be cut by a knife, like chalk or graphite, and its specific gravity is only about 2.3. Some cases have been found where this condition extends entirely through the shell of the pipe; in others it extends from one-eighth to three-eighths of an inch in depth. This deterioration is not uniform in the same pipe; some of the pipes, in which the metal has in places changed through the entire thickness, having other places where no perceptible change has taken place. In some spots the tar coating is still to be found, but for much of the outer surface of the pipe, it has entirely disappeared."

Later, when the speaker succeeded Mr. Hawley in office, he found the conditions much the same, although no point was observed where this alteration extended through the entire thickness of the pipe. From a general consideration of these facts, the speaker would estimate 25

years as the extreme limit of the time during which dependence could be placed on cast-iron pipe under similar conditions, and is of the opinion that steel pipe coated as this was, with two layers of burlap and mineral rubber, is not suitable for use. Mr. Allen.

It may be of interest to add that while a sewer trench was being excavated along the water-front in Baltimore, several cannon balls varying from 4 to 6 in. in diameter were found several feet below tide-level, and, as there had formerly been a landing at this point, which subsequently was filled in, it seemed probable that these had been dropped from some vessel during the War of 1812. In spite of having remained in this position so many years, they were practically free from rust.

WILLIAM W. BRUSH,* M. AM. SOC. C. E.—New York City has more than 2 500 miles of pipe in its distribution system, and has had practically no difficulty from external corrosion. An examination, made about a year ago, of a length of 48-in. pipe which was laid in 1867 in Central Park, near the large reservoir, in ground that was low-lying and wet a good part of the year, showed the outside coating to be practically perfect. The general condition of the entire pipe system of New York City is such that the outside of the pipe generally shows no corrosion that would affect its life. Mr. Brush.

The interior of the pipe shows very materially the difference in effect of waters supplied to the different boroughs of New York City. Croton water has such slight corrosive effect on cast-iron pipe that sections of mains laid shortly after this water supply was introduced, in the early Forties, or in use some 60 years, show practically no corrosion. Therefore, there has been no necessity for cleaning these mains; whereas in Brooklyn, where the water has a comparatively high chlorine content, varying during the past 50 years from a maximum of about 25 parts per million to a minimum of about 8 parts, with an average for the past 20 years of possibly 15 parts per million, there is a corrosive or tuberculating effect, especially on pipe laid uncoated. The pipe laid in 1858 was mainly Scotch pipe, in 9-ft. lengths, and was uncoated. There was also some pipe cast in the United States at about that time and laid uncoated. Such pipe required cleaning, as the tubercles in a 6-in. pipe were of such size that a 2-in. rod could not be passed through the pipe without breaking off some of them. That pipe, when cleaned, was good, apparently, for another 50 years, as far as the strength of the wall limited the life. The rusting probably advances rapidly on the cleaned pipe. Thus, at Far Rockaway, pipe which had been laid from 20 to 30 years ago, and was cleaned about 10 years ago, now shows tubercles of a depth and volume equal to those found in pipe laid some 20 or 30 years

* New York City.

Mr. Brush. ago, which has not been cleaned. Apparently, with that particular water, tubercles developed in 10 years after cleaning, which would have taken 20 or 30 years to develop in the pipe as originally laid.

It is very important, however, to determine under what conditions rapid corrosion of the iron will take place on the exterior of the pipe. In many cases, even where protection would cost large sums, the cost of external protective measures for pipe lines would pay. On the other hand, there are cases where such protection would not pay.

Experience in New York City, where the pipes are frequently laid in ground-waters, especially along the water-front, indicates that there is no condition that would warrant any special method of protection.

Mr. Wagner. SAMUEL TOBIAS WAGNER,* M. AM. SOC. C. E.—The author has referred to pipe laid in Philadelphia in 1817. In 1897, about 1 000 ft. of this pipe, 22 in. in diameter, was removed from the bed of Pennsylvania Avenue on account of street changes. After 80 years of service, its external condition was almost perfect. It had been laid in a sandy and fine gravelly soil, on ground well above tide-water. The full section of the metal in the pipe was there, and although tubercles were rather freely distributed on the inside, the loss of waterway section was only about 4.2 per cent. The metal inside was generally in excellent condition.

Reference has been made to the effect of chlorine in the water on the internal condition of a cast-iron pipe. This is a new thought to the speaker, and for information, it might be interesting to add that the water in the Schuylkill River, which passed through this pipe line, has a chlorine content of about 5 to 6 parts per million.

The data presented by the author are of great interest, and serve to accentuate the necessity of studying local conditions before using any metal when corrosion is to be feared. There are many places where cast iron is the very best material that can be used to resist corrosion, and it would appear that the author believes that water pipe, when not exposed to the action of salt water, is one of them. Some cases of the resistance of this metal to ordinary atmospheric conditions are remarkable. In some of these, cast iron is still perfect where wrought iron and steel, similarly exposed, have rusted away. There can be no doubt that the resistance of the skin coat of silicious material is one of the chief reasons for its durability.

Mr. Kellogg. R. C. KELLOGG,† JUN. AM. SOC. C. E.—There has been so much advertisement and so much written in regard to the everlasting life of cast-iron pipe that information such as contained in this paper is of value, in order that the exceptions to the general rule as to the longevity of such pipe can be taken into consideration when using it.

* Philadelphia, Pa.
† Brooklyn, N. Y.

Mr. Kellogg.

Some years ago the speaker was familiar with conditions of quite severe pitting on a 60-in. riveted, steel main laid from Astoria to Ravenswood, Queens County, New York, to convey gas from the new Astoria Works to the Ravenswood plant. At the time, these pittings were attributed to electrolytic disturbances, but, in view of the information presented in this paper, it is only fair to assume that this was an error, and that the major causes were tidal action and the soil in which the main was laid, which was originally part of a salt marsh. This idea is somewhat strengthened when the speaker recalls the fact that an electrolysis survey did not seem to develop sufficient evidence that electrolysis was the sole cause of the trouble. He is familiar with several cases in Brooklyn where cast-iron gas mains have been taken up after having been in the ground from 50 to 60 years, such mains showing no signs of exterior corrosive injury. There are also several cast-iron gas mains crossing the Harlem River, at various points, all of which are covered by salt water and mud. Parts of these mains have been taken up, on account of accidental breakage, and show absolutely no effect of corrosive action.

From this it would appear that corrosion is caused, not only by soil of a certain character, and tide-water, but also that the air has an appreciable effect. The speaker is not familiar with any cases of internal corrosion, because his experience has been entirely with gas pipes, in which the interior surfaces are preserved by the deposition of liquid hydro-carbon compounds condensed out of the gas, due to friction and changes in temperature.

Mr. Flinn.

A. D. Flinn,* M. Am. Soc. C. E.—Some years ago, while visiting one of the yards of the Spring Valley Water Company, in San Francisco, the speaker was shown several lengths of wrought-iron pipe which had been taken from the ground after 20 years' service and was in perfect condition, both inside and outside. The pipe had been made in accordance with specifications by the Chief Engineer of the Company, and the asphalt coating had been made and applied also according to these specifications, the work being done with great care in all details. The speaker was informed by the engineer who accompanied him that pipe was frequently taken up, where changes in the system were necessary, and found to be in such excellent condition that, when necessary, it was re-used.

In connection with the Catskill Aqueduct, in laying pipe across The Narrows, from Brooklyn to Staten Island, a search was made for a coating that would be a little more durable than most of the known pipe coatings, and some inquiries gave hopeful accounts of one known in the trade as "bitumastic enamel". The pipe line, on

* New York City.

Mr. Flinn.

which this coating is being used, is now being laid, and the work will probably continue during the fall and next spring.

In respect to galvanizing as a means of protecting steel and iron, experience on the Catskill Aqueduct is encouraging. Steel-plate manhole covers, railings, and other objects which have been galvanized and exposed to the weather, have remained in good condition.

Mr. Boucher.

WILLIAM J. BOUCHER,* ASSOC. M. AM. SOC. C. E.—The use of cast-iron pipe is so general that the speaker believes that any data relating to the subject of corrosion will be of interest, perhaps, of value.

Until 1897 or thereabouts, Jersey City, N. J., was supplied with water from the Passaic River. The water was pumped from a station at Belleville, opposite North Newark, into and through a 36-in. cast-iron main laid alongside the Turnpike Road to a reservoir on Bergen Hill, Jersey City. This pipe was made by the Warren Foundry and Machine Company, and the letters, "W. F. & M. C., 1862", are still discernible to anyone with good eyesight. The pipe is supported, as described later, practically at the surface of the grass-and-weed-covered tidal salt marsh, which extends from the west side of Bergen Hill about 7 miles to the east bank of the Passaic River. It was laid during 1862 and 1863 and, with the exception of a few months in 1898, has conveyed water throughout its whole length since 1863. It shows no signs of pitting or disintegration, and is in practically a perfect state of preservation. With a pocket knife, the speaker removed a few patches of covering resembling asphalt. These patches covered only a minute fraction of the entire surface, and if the pipe was at one time entirely covered with a preservative, it has long since ceased to be of any value as the pipe, everywhere, is uncovered and the upper portion of its circumference plainly shows that it is used frequently as a foot-path.

The following description of the construction, etc., taken from the "Report of the Engineer of the Board of Water Commissioners, 1862-63," may be of interest:

> "Two rows of piles are driven, three feet apart, to a solid foundation, which is found at a depth varying from 12 to 20 feet below the meadow surface. The heads of the piles are then cut off 16 inches below the meadow and cross capped with 10 x 12-in. pine timber; upon this capping a brick pier 12 inches thick, 44 inches long, and 13 inches high, is laid up in hydraulic cement.
>
> "The pipe when placed upon this pier is securely held in its position by additional courses of brick work built up on its sides. The piling and capping are thus kept constantly immersed in water, rendering it perfectly secure from decay, and the joints of the pipe being just above the meadow surface enable them to be caulked without difficulty."

* New York City.

Mr. Boucher.

The pipes are 36 in. in diameter, $1\frac{1}{20}$ in. thick, and 12 ft. 5 in. long, averaging 4 800 lb. in weight. During 1862, 22 400 lin. ft. of completed main were laid, at a cost of:

Iron pipes at $37 per ton, delivered.....	$138 121.30
Contract for laying pipes, including piling, capping, brick foundations, and covering pipes with earth.	46 567.20

The object of the earth covering was to prevent the freezing of the water during the winter, but, according to the speaker's recollection, the pipe has had no earth covering for 20 years, and was conveying water when observed in October, 1914.

Mr. Purver.

GEORGE M. PURVER,* ASSOC. M. AM. SOC. C. E.—The speaker would like to ask Mr. Pugh, if he had any opportunity to observe the rate at which rusting of cast-iron pipe takes place. From the time a pipe begins to rust, does the rate of rusting remain uniform and vary directly with the time of exposure, or not; and would the author expect the same results under similar conditions?

The speaker believes that this paper should prove interesting and instructive to every member of the Profession, and it has suggested to him the idea that it would be desirable to urge the Society to appoint a "Special Committee on Deterioration of Structural Materials," such as stone, concrete, steel, cast iron, and wood.

Such a committee could gather all the data available on rusting of metals, rotting of wood, disintegration of stone and concrete, etc., and plot curves showing periods of time of their reliable service. It is possible that, with the assistance of coefficients established from observation of the conditions surrounding the material in use, one would be able to approximate the rate of deterioration.

The speaker does not recollect a single volume or pamphlet on this important subject that would comprise all the reliable data on the life of various structural materials, and yet in preparing an estimate or in evaluating an engineering enterprise, such data would furnish an important element.

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* Brooklyn, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE CLARIFICATION OF SEWAGE BY FINE SCREENS.

Discussion.*

By Messrs. G. Bertram de B. Kershaw, W. L. Stevenson, George C. Whipple, C. A. Jennings, G. Franze and — Schaefer, George W. Fuller, J. X. Cohen, F. T. Robson, Alexander Potter, and Charles E. Gregory.

Mr. Kershaw.

G. Bertram de B. Kershaw,† M. Am. Soc. C. E. (by letter).—The first point noted, in reading Mr. Allen's valuable paper, is that it deals almost exclusively with large cities and their sewerage installations.

It is suggested in the paper that one of the conditions where fine-screening should generally be considered is "where the removal of all but the very fine solids will furnish a satisfactory effluent." It follows from this that the city contemplating fine-screening as a final process must have a suitable and sufficient body of diluting water available to carry out the purification process proper by oxidation, effected by abstraction of dissolved oxygen from the diluting body.

Conditions in England (where almost all the writer's first-hand experience has been obtained), are, as a general rule, unfavorable for the adoption of processes which throw the brunt of the purification on the body of water into which the sewage liquor is discharged.

English rivers, with very few exceptions, are small, in marked contrast to many of the rivers of the Continent; moreover, many of the smaller rivers receive sewage effluents at short intervals of distance throughout almost their entire courses, and are not in a condition to cope with any fresh burden of self-purification.

It is found to be the case, therefore, that screening is almost entirely confined to what may be termed "coarse" screening, that is, a screening

* This discussion (of the paper by Kenneth Allen, M. Am. Soc. C. E., published in August, 1914, *Proceedings*, and presented at the meeting of October 21st, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† West Wickham, Kent, England.

Mr. Kershaw.

process only intended to remove the grosser solids, such as sticks, stones, coal, corks, potato peelings, and the like, which otherwise would be likely to lodge under sluice-valves and penstocks, and cause trouble; screens are also provided when the sewage is subsequently pumped, or, to protect the filter presses, where sludge pressing is carried out.

Such screens are generally of the bar type, cleaned by hand or automatic rakes, the openings between the bars being from about ½ in. to 1 in., and greater.

As giving a good idea of a very usual type of rough screen used at many sewage works in England, the following particulars regarding the Stratford-on-Avon sewage works (as being typical of a small English country town) may be of interest.

The screen at the pumping station is of ⅝-in. bars, spaced ⅝ in. apart, and it is inclined at an angle of about 20° with the perpendicular. Cleaning is effected by a system of traveling rakes attached to two endless chains, which elevate the screenings and tip them into a tumbril cart. It should be observed that the Stratford sewage is a strong one and contains a good deal of brewery waste.

Table 14 gives particulars relating to the screening operations.

TABLE 14.—ANNUAL QUANTITY OF SCREENINGS PRODUCED AT STRATFORD-ON-AVON SEWAGE WORKS.

Year.	Total yearly sewage flow, in millions of gallons.	Tons of screenings.	Tons, per million gallons.	Cost of Removing Screenings.		Rainfall for the year, in inches.
				Per million gallons.	Per ton.	
1908–09	126	100.5	0.70	1s. 3d.	1s. 7d.	21.06
1909–10	134	103.5	0.77	1s. 1d.	1s. 3¼d.	26.95
1910–11	136	99.0	0.73	5.9d.	8.1d.	25.17

Where special trade sewages have to be dealt with, such as those containing wastes from wool combing or the cotton industry, screening forms an important integral part of the sewage plant, but it should be observed that such screening is only to be regarded as ancillary or preliminary to the purification process proper, and it is intended merely to remove substances which might cause mechanical troubles at a later stage, reliance being placed on subsequent processes for real purification. It is, in fact, a process of abstraction rather than of purification, as generally understood.

The writer believes he is correct in saying that there are very few, if any, screening plants in England which are designed to effect other than a partial withdrawal of the larger suspended solids, such as are peculiarly likely to float and give an unpleasant appearance to the water into which they happen to be discharged. Certainly there are no

fine-screening plants capable of producing the results mentioned by Mr. Allen as being achieved in Germany by the best examples of fine screens. Mr. Kershaw

Fine-screening would appear to be peculiarly adapted to seaside resorts in England, where bathing is enjoyed.

The Royal Commission on Sewage Disposal,* for which the writer has been Engineer for some sixteen years, in dealing with standards, states as follows:

"But we think it of great importance, if only for the sake of the appearance of the river, that the grosser solids, such as undisintegrated fæces, paper, etc., should, as far as possible, be held back, and the provision of some effective form of screening apparatus should be insisted on wherever practicable."

There is no doubt that, when the treatment of sewage by dilution is better understood in England, fine-screening will become of considerable importance. Screening as a final process in certain cases is likely to be carried out when the Commission's reports have been digested by the English Local Government Board. At present, there is some uncertainty as to how "dilution" schemes would be received officially.

The following statement shows the degree of dilution and limiting figures for suspended solids in sewage liquors, laid down by the Commission for cases where disposal by dilution is to be countenanced.

Dilution.	Suspended solids, in parts per 100 000.
Exceeding 150 volumes and less than 300 volumes.....	6
Exceeding 300 volumes and less than 500 volumes.....	15
Exceeding 500 volumes.....	Crude sewage could be discharged subject to such conditions, as to the provision of screens or detritus tanks, as might appear necessary to the central authority.

It may be noted that the following figures show what would be considered good tank liquors in England, assuming a domestic sewage of average strength, containing some 35 parts of suspended solids per 100 000, to have been treated.

Treatment.	Parts per 100 000.
Septic tank treatment.....	10 to 15
Continuous flow settlement.....	10 to 15
Quiescent settlement	5 to 8
Continuous flow precipitation.....	3 to 6
Quiescent precipitation	1 to 4

* Eighth Report, Royal Commission on Sewage Disposal, p. 12, para. 42.

Mr. Kershaw.

It should be observed that, for the determination of the suspended solids, a Gooch crucible, packed with asbestos floss, is used.

An English domestic sewage of average strength contains about 35 parts of suspended matter per 100 000, but the quantity is variable. A large part of these solids is usually very finely divided. The actual fineness will depend to a great extent on the sewerage system, the distance the sewage has traveled, whether pumped (which often breaks up the solids and increases colloidal matter) or flowing by gravitation, nature of street surfaces, if the combined system of sewers is in use, temperature, and many other variable factors; but a large proportion of the finer suspended solids will almost invariably pass with ease through a mesh of $\frac{1}{50}$ in. or even less; and, with a septic sewage, such as sometimes results in hot weather when the sewage has traveled far, the finest screens will effect very little useful work.

In most sewages, trade wastes of various kinds form a feature, and the suspended solids in the sewage may then be very greatly increased, and vary from hour to hour. Again, a population served entirely by water-closets will, as a rule, contribute much more screenable matter than where the whole of the fæces and paper does not enter the sewers.

The writer has followed the Chicago sewerage problem for some time past, and the percentage of removal effected by Mr. Jennings' screen (63%) seems very high, notwithstanding the fineness of the mesh (40 per in.). Possibly it is accounted for by the nature of the sewage, which presumably contains suspended matter of a uniform nature and size, such as masticated hay, etc. If this is so, such screenings would be likely to resist decay, owing to the large quantity of cellulose matter they would contain.

The nature of the suspended solids in a sewage varies considerably. From 40 to 60% of the suspended matter in most English sewages consists of mineral or inorganic matter, practically innocuous by itself (excepting as regards the silting up of waterways), but it is found that these particles carry with them, as a kind of envelope, a considerable quantity of organic impurity in solution, and they exercise a de-oxygenating action on rivers and streams, albeit not such a sustained action as solids of organic origin.

When dilution is relied on to complete the purification of a settled or screened sewage liquor, the main essential is to keep the bulk of the suspended matter in suspension in the river water until it has become oxidized or nearly so.

The capacity of a clean river to endure pollution without visible signs of nuisance appears to be closely related, among other things, to its depth, velocity, power of transportation, temperature, and aquatic plant, fish, and insect life.

Mr. Kershaw.

If the mud near the borders of a river receiving a certain quantity of pollution be examined, it will generally be seen to consist of very fine particles of black mud, overlaid by a thin layer of brown (or nearly oxidized) mud on the surface, where it has been intimately in contact with the dissolved oxygen in the river water. How fine much of this mud is, may be judged by mixing some of it with tap water and straining the liquid through a handkerchief into a glass; much of the fine matter will be found to pass through the linen with the water.

It is mainly these very fine solids which cause de-oxygenation troubles in sluggish rivers or sluggish reaches of rivers, when they become deposited, and it is very difficult, with good tank treatment, to keep them back, unless precipitants are used.

In England, however, there are instances where, owing to favorable local conditions in a river—such as volume and velocity—such fine suspended matter might be discharged without any perceptible damage to the river, and in such cases fine-screening of the nature indicated by Mr. Allen would be of marked value.

The writer has seen examples where the removal of the tank effluent outfall from the bank of a river to a point some distance from the shore would have admitted of fine-screening in lieu of tank treatment. It is quite possible to foul a strip of river seriously by the discharge of tank liquor or sewage close to the shore, whereas the admixture of screened sewage with the whole volume of river water would suffice for purification purposes.

With small rivers in England, the writer has no doubt that less damage would result to the rivers by passing fresh sewage from a small community into them, after efficient screening, than, as is often done, by passing the sewage through a septic tank and thence to the river. Septic tank treatment almost invariably results in the production of sulphuretted hydrogen, which has the property of taking up dissolved oxygen from water very rapidly. An Imhoff tank would probably solve such problems, but screening would be cheaper.

As Mr. Allen carefully points out, screenings consist largely of organic matter, and considerable weight must be attached to the fact that, with suitable forms of screens, organic matter, such as fæces, etc., are removed at once from contact with the liquid portion of the sewage, whereas, with tank treatment, much of this organic matter is left in contact with the liquid in the tank for some considerable time, more and more of it going into solution as fermentation proceeds.

On the other hand, screenings are more likely to attract dipterous insects for ovipositing than sludge; still, they might be deterred by the use of bleaching powder or some similar chemical. Both screens

Mr. Kershaw.

and screenings should be roofed in wherever possible, for the above reason, quite apart from danger of nuisance from smell.

The results obtained by fine-screening in Germany, cited by Mr. Allen, appear to show an extraordinary efficiency, and it is difficult to imagine such a percentage as 90.

Experiments carried out at Leeds, England, with a fixed-mesh screen of 30 per in., inclined with the sewage flow, appeared to keep back a considerable proportion of the suspended matters in the sewage, but "although the accumulation seemed bulky, it was found on drying to represent less than 10% of the suspended matter in the sewage." (Leeds sewage contains about 60 parts of suspended matter per 100 000.) It may be noted that the clear liquid of a sample of average Leeds sewage which had been passed through filter paper still contained in solution about 50% of the original impurities, becoming dark-colored and putrefying.

There can be no question that screenings and sludge stand on an entirely different footing as regards manurial value and fat content. Screenings consist largely of nitrogenous matter, and usually contain little mineral matter; whereas, in sludge the solids are generally 50% inorganic, and it is worth just what can be obtained for it.

The fat content of the screenings at Hamburg is well brought out in Table 1, showing the composition of grit and screenings, the average being nearly 5 per cent. In Table 2 it would be interesting to know whether the nitrogen and phosphates are in as readily available form for plant life as in artificial manures. The manurial constituents of most sewage sludges appear to be in an inert and slowly-acting condition, and the writer has always considered that a great part of any of the value that sewage sludge may possess, is due to its physical and mechanical, rather than its manurial, action on soils.

Regarding the Carshalton screen, it should be observed that all these screens are used for either crude or grit-settled sewages. The writer does not know of any plants in which they are used for screening tank liquors. It may be noted that this screen is driven by a water-wheel actuated by the flow of sewage, and the speed at which the screen travels is automatically adjusted to the work required of it.

This is a matter of importance, because, if a screening apparatus reduces the velocity of the sewage flow beyond a certain point, settlement of suspended matter will take place up stream from the screen; it is of importance, therefore, for automatic screens to work at a rate which will allow the sewage flow to be maintained at a velocity sufficiently high to keep the solids in suspension. If, owing to too slow a motion of the screen, or temporary insufficient area of mesh openings (such as might result from paper flattening against the meshes), the decrease in velocity causes the sewage to head up, and, consequently,

adjacent storm overflows may be brought into operation, especially if the gradient of the main outfall sewer is a slack one. Mr. Kershaw.

It is a most complicated matter to arrive at the real efficiency of screens, and this is largely due to the difficulty of obtaining reliable average samples, and of procuring accurate analytical figures covering a long period of experimentation.

The accurate determination of suspended solids is a tedious process, and it might be possible to use a centrifuge in order to save time. If this was done, however, results obtained with a sewage containing suspended solids of a granular nature would scarcely be comparable with those obtained elsewhere with a sewage containing flocculent suspended solids, although they would be reliable for the one place if the suspended solids were constant in character.

The most important point appears to the writer to be, not how great a quantity of screenings is removed per million gallons of sewage, but how much suspended matter is left in the screened liquor. A screen may appear to remove a large quantity of matter from the sewage (as in the Leeds experiments), and yet, on carefully analyzing the screened liquor, the quantity of suspended solids is usually found to be high.

It is of vital importance, therefore—and it is noted that Mr. Allen lays stress on this point—that, in any screening experiments, the quantity of suspended solids, both in the raw sewage and in the screened liquor, should be most accurately determined, and for a considerable period, during both hot and cold weather. During cold weather one would expect more screenings to be removed than in hot weather, and, turning to the Dresden results, given in Table 10, if the weight of dry material in the screened liquid be subtracted from the weight of dry material, as in Table 15, it will be seen that the matters removed by screening reached a maximum in the coldest weather, and this is what one would have expected, because fermentation and its power of breaking down organic matter is then at a low ebb.

TABLE 15.

Period.	Weight of dry material in raw sewage, in grammes.		Weight of dry material in screened liquid, in grammes.	Weight of dry material removed by screens, in grammes.
March, April, May	50.96	less	41.40	= 9.56
June, July, August	61.42	"	50.78	= 10.64
September, October, November	81.71	"	59.13	= 22.58
December, January, February	99.41	"	51.33	= 48.08

It is essential, in any comparative trials of screens, that all samples should be average ones, that is, drawn every hour or oftener throughout

Mr. Kershaw.

the 24 hours, according to the rate of flow of the sewage, otherwise very misleading results may follow; further, the samples of raw sewage and of screened sewage should be strictly corresponding ones; these can be ensured as a rule by the use of a coloring dye, such as fluorescin (green) or eosin (red).

In connection with this last point, the figures of analysis given in Table 4 for chlorine, show that the samples were not corresponding ones, the figure for the raw sewage being 262.6, and that for the screened sewage, 279.4; these figures should have been practically identical. It is to be noticed that there is no material difference in the polluting property of the raw and screened sewages, judged by the figures for "oxygen consumed."

Regarding the results given in Table 4 with reference to the Geiger screen, it appears probable that the rolling action of the vanes, as the screenings pass to the axis, tends to comminute, or perhaps to send into solution, some organic matter, thereby increasing the strength of the screened sewage.

Some methods of cleaning screens leave much to be desired, from this point of view, and sewages are sometimes stronger as regards dissolved impurities after screening than before, from this cause. Good management must always be forthcoming, if the best results are to be obtained.

In order to gain a strictly scientific comparison between various screens, it would seem that the only way would be to test them all on the same sewage; even this procedure might not hold good, because a particular type of screen might be better adapted to some sewages than to others.

In any case, trials of screen efficiency carried out with one sewage would be costly, because they would need to be tried on a working scale to yield useful results.

A point needing special attention would be the moisture contained in the screenings, and it would seem advisable to devise a standard method of draining, whereby the screenings in each test would be drained for a given time on a specially formed drying area before being sampled for analysis; moreover, the sub-sampling and sampling of the screenings would require very great care.

The hourly sewage flow and the velocity through the screens would need to be carefully recorded in all trials, because solids are apt to be forced through even fine screens under conditions of heavy sewage flow.

Table 12, summarizing the data relating to the screens discussed by Mr. Allen, is valuable, and the cost of operation does not anywhere appear to be excessive; it is to be observed, however, that the whole question of efficiency depends on the sampling and analysis, and the accuracy with which these are done.

Mr. Kershaw.

Will Mr. Allen state whether the figures in Table 12 for cost of operation per million gallons include interest on capital outlay, or depreciation? The writer is of the opinion that they do not.

It would have been most interesting if the author could have given columns showing the volumes of water into which the screened liquors were discharged, and the visual effect produced on the water and banks. The writer is well aware, however, of the difficulties involved in obtaining reliable information on these points.

The writer is in general agreement with Mr. Allen's conclusion, although rather doubtful whether a 30% removal of total suspended solids from an average domestic sewage is not too sanguine an estimate to be uniformly attained.

Mr. Allen is to be congratulated on a most interesting and instructive paper. A vast amount of time and labor must have been expended in collecting and assorting the minute data it contains, and it would be well if more engineers in England would follow Mr. Allen's example and go and see for themselves anything new connected with sewage purification and disposal plants.

A feature of this paper is the excellence of the half-tones with which it is illustrated.

Mr. Stevenson.

W. L. STEVENSON,* Assoc. M. AM. SOC. C. E. (by letter).—During 1909-10, the Bureau of Surveys of Philadelphia operated a sewage testing station in connection with the investigations being carried on for the preparation of a comprehensive plan for the collection, treatment, and disposal of the sewage of that city.

Among the processes studied was fine-mesh screening. The observations throw light on the second and third functions of fine screens mentioned by Mr. Allen, and also on the character of the sludge produced in subsequent sedimentation, and the efficiency of fine-screening as a preparatory treatment for subsequent disinfection.

The sewage used in the testing station was collected by a separate system of sewers from a territory partly residential and partly devoted to the textile industry, and contained a considerable quantity of trade wastes.

The fine-mesh screen consisted of a cone covered with red metal cloth, 32 meshes per inch, and having clear openings ½ mm. square. Sewage which had been removed from the side at about mid-depth of the flow in the intercepting sewer was raised to the testing station by a plunger pump and applied to the screen through twenty-four ¼-in. nozzles. No attempt was made to design a screen for practical use, the screenings being washed away to a trough by the splashing of the jets; but screened sewage was obtained for subsequent study. On an average, the sewage applied to the screen contained 200 parts

* Philadelphia, Pa.

Mr. Evenson.

per million of suspended matter, as measured by the Gooch crucible; and the average effluent contained 133 parts per million. The functions of this screen on subsequent treatment of the effluent were as follows.

"2. The Removal of Solids That Otherwise Would Cause the Frequent Subsequent Clogging of Sprinkler Nozzles or Filters."—The screened sewage, after plain sedimentation in a horizontal tank, was applied to a percolating filter through two square spray Taylor nozzles. During 9½ months service, these nozzles were never clogged; whereas, the same crude sewage after similar sedimentation without fine-screening caused frequent nozzle-clogging from match sticks, wool fibers, etc. The interstices of the media in the filter to which the screened and settled sewage was applied, were invariably cleaner and freer from sludge than those in filters to which only settled crude sewage was applied.

"3. The Removal of Material Likely to Promote the Formation of Scum in Subsequent Tank Treatment."—Two horizontal-flow, sedimentation tanks, each provided with baffle- and scum-boards of identical construction and operated at the same rates, received as influents, respectively, crude sewage and the effluent of the fine screen. In the tank receiving crude sewage a scum always formed a few days after it was put in service, and before cleaning the tank it usually attained a thickness of more than 1 ft. It was composed of a leathery mat of wool fibers, hops, and other sewage matters which were light enough to float.

The surface of the tank receiving the screened sewage always remained clean and free from any floating matters. This had an effect on the quality of the effluent. Septic action developed much sooner in the tank receiving unscreened sewage, and the effluent was more deoxidized than that from the tank receiving screened sewage.

Naturally, the effluent of the tank receiving screened sewage was lower in suspended solids than the other, and its quality was also more uniform. Sudden increases in suspended solids in the crude sewage produced a synchronous increase in those in the effluent of the tank receiving crude sewage; but no change was noted in the effluent of the tank receiving screened sewage, as the screen relieved the tank from these peak loads of suspended matter.

The Effect of Fine Screening on Sludge Formed in Subsequent Tank Treatment.—On an average, 4.07 cu. yd. of sludge of 1.053 specific gravity and containing 86.1% of moisture, were deposited from each 1 000 000 gal. of crude sewage settled, and 4.65 cu. yd. of sludge of 1.036 specific gravity and containing 90% of moisture, were deposited from each 1 000 000 gal. of screened sewage settled. The lower specific gravity and higher water content of the sludge from the screened sewage was due to the fineness of the particles of which

Mr. Stevenson.

it was composed. Each cubic yard of the crude sewage sludge contained, on the average, about 990 lb. of dry residue and 790 lb. of the screened sewage sludge.

The effect of the fine screen, therefore, was to increase the volume of sludge by 14%, but to reduce the quantity of dry solids deposited by 20 per cent.

The screened sludge, however, dried more readily than the crude sludge, due to absence of wool fibers which prevented the formation of deep cracks in the latter. When removed from the drying beds, the quantity of screened sewage sludge was less than that of crude sewage sludge.

Efficiency of Fine Screening as a Preparatory Process to Disinfection.—Experiments were carried on to determine the quantity of calcium hypochlorite required to disinfect fine-screened sewage, settled crude sewage, and settled screened sewage. The results were divided into two groups: First, a quantity of disinfectant which should be added during an epidemic of intestinal disease to insure protection to communities below the point of discharge; and, second, a quantity which would destroy more than 95% of the *B. Coli* at a moderate cost. The results are given in Table 16,

TABLE 16.

Kind of sewage.	Available chlorine added, in parts per million.	Total Number of Bacteria per Cubic Centimeter, on Gelatine at 20° Cent., in 48 Hours.			*B. Coli* per Cubic Centimeter, as per Jackson's Presumptive Test.		
		Initial.	Final.	Percentage remaining.	Initial.	Final.	Percentage remaining.
Fine-screened sewage....	12.4	2 470 000	337	99.99	121 000	20	99.98
	6.0	2 060 000	181 000	91.21	149 000	7 470	95.42
Settled crude sewage....	11.9	2 450 000	350	99.99	143 000	10	99.99
	5.4	760 000	31 000	95.92	67 000	745	98.89
Settled screened sewage.	12.0	2 130 000	310	99.99	86 000	20	99.98
	4.3	660 000	22 500	96.59	317 000	1 350	99.57

and show that sedimentation of the sewage was more efficient as a preparatory process to disinfection than fine-screening; but that, in cases where the receiving body of water is capable of assimilating, without nuisance or detriment, the effluent of a fine-mesh screen containing particles not larger than 1 mm., it is practical to disinfect it. Experiments have been made to show that it is not practical to disinfect sewage containing particles larger than 1 mm.

Summary.—The results of the experiments made by the Bureau of Surveys show that the use of a screen of 32 meshes per inch prior to sedimentation produces an effluent which will not clog the nozzles

Mr. Stevenson.

or media of a percolative filter; prevents the formation of scum in subsequent tank treatment; causes a uniform quality of tank effluent, regardless of irregular quantities of suspended matter in the crude sewage; yields a sludge from subsequent sedimentation higher in moisture and of greater volume, but containing less dry solids, than from the sedimentation of crude sewage; and is a satisfactory preliminary treatment for subsequent disinfection.

These data are merely statements of facts. The application of fine-screening of sewage to any particular case is an engineering problem to be determined by the local needs and conditions.

The writer wishes to express his appreciation of the valuable data which Mr. Allen has furnished in this paper.

Mr. Whipple.

GEORGE C. WHIPPLE,* M. AM. SOC. C. E. (by letter).—This is an excellent paper on sewage screening, and should form the nucleus for some interesting discussions. With the author's main contention, that fine-screening is destined to play an important part in sewage treatment in America, the writer is in hearty sympathy. That was the conclusion which he reached a number of years ago after a trip to Europe which was referred to in a discussion presented to the Society in June, 1906.

In the prevention of the pollution of streams, lakes, and harbors it is necessary to have laws which govern the discharge of sewage and trade wastes into such bodies of water. These laws must not be discriminatory, but, in the interest of fairness, must be general in their application, and yet each city or town must be permitted to take charge of its local facilities for sewage discharge, using them to the limit, but with due regard to proper sanitation. Undoubtedly, there are local situations where the discharge of raw sewage into streams will do no harm, but it is difficult to frame and execute laws which will fairly separate the places where no sewage treatment is needed from those where treatment of some kind is necessary. From an administrative point of view, it may be better to make the law general that no raw sewage shall be discharged into our waterways, and leave the nature of the treatment in each case to some properly constituted authority. In doing this it will be necessary to give a broad meaning to the term "sewage treatment" and allow it to cover, not only the more elaborate processes of purification, but also the simpler methods of coarse screening.

Between raw sewage and that which has passed through coarse screens there is no material bacteriological difference, and we might almost say the same with reference to raw sewage and that which has passed through the fine screens described by the author. Yet the process of screening is so inexpensive, and it does so much to

* Cambridge, Mass.

Mr. Whipple.

avoid the obvious signs of pollution by the removal of the lighter and the grosser solids, that there are few cases where the process is not justified by its results. By giving this liberal interpretation to the expression "sewage treatment" it will be easier to obtain legislative action relating to the control of sewage discharge. The engineers who recently advised the International Joint Commission, in the matter of the control of sewage disposal for communities on the boundary waters between the United States and Canada, took this view when they formulated the following guiding principles. It will be seen that they were worded so as to include screening as a form of treatment. For this reason they may be appropriately inserted in this discussion.

"NEW YORK, *June 27, 1914.*

"To the INTERNATIONAL JOINT COMMISSION.

"GENTLEMEN: The following statements represent the essence of the opinions given by us before your honorable commission at the conference held in New York on May 26 and 27, 1914:

"1. Speaking generally, water supplies taken from streams and lakes which receive the drainage of agricultural and grazing lands, rural communities, and unsewered towns are unsafe for use without purification, but are safe for use if purified.

"2. Water supplies taken from streams and lakes into which the sewage of cities and towns is directly discharged are safe for use after purification, provided that the load upon the purifying mechanism is not too great and that a sufficient factor of safety is maintained, and, further, provided that the plant is properly operated.

"3. As, in general, the boundary waters in their natural state are relatively clear and contain but little organic matter, the best index of pollution now available for the purpose of ascertaining whether a water-purification plant is overloaded is the number of *B. coli* per 100 cubic centimeters of water expressed as an annual average and determined from a considerable number of confirmatory tests regularly made throughout the year.

"4. While present information does not permit a definite limit of safe loading of a water-purification plant to be established, it is our judgment that this limit is exceeded if the annual average number of *B. coli* in the water delivered to the plant is higher than about 500 per 100 cubic centimeters, or if in 0.1 cubic centimeter samples of the water *B. coli* is found 50 per cent. of the time. With such a limit the number of *B. coli* would be less than the figure given during a part of the year and would be exceeded during some periods.

"5. In waterways where some pollution is inevitable and where the ratio of the volume of water to the volume of sewage is so large that no local nuisance can result, it is our judgment that the method of sewage disposal by dilution represents a natural resource and that the utilization of this resource is justifiable for economic reasons, provided that an unreasonable burden or responsibility is not placed upon any water-purification plant and that no menace to the public health is occasioned thereby.

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"6. While realizing that in certain cases the discharge of crude sewage into the boundary waters may be without danger, it is our judgment that effective sanitary administration requires the adoption of the general policy that no untreated sewage from cities or towns shall be discharged into the boundary waters.

"7. The nature of the sewage treatment required should vary according to the local conditions, each community being permitted to take advantage of its situation with respect to local conditions and its remoteness from other communities, with the intent that the cost of sewage treatment may be kept reasonably low.

"8. In general, the simplest allowable method of sewage treatment, such as would be suitable for small communities remote from other communities, should be the removal of the larger suspended solids by screening through a one-fourth inch mesh or by sedimentation.

"9. In general, no more elaborate method of sewage treatment should be required than the removal of the suspended solids by fine screening or by sedimentation, or both, followed by chemical disinfection or sterilization of the clarified sewage. Except in the case of some of the smaller streams on the boundary, it is our judgment that such oxidizing processes as intermittent sand filtration, and treatment by sprinkling filters, contact beds, and the like, are unnecessary, inasmuch as ample dilution in the lakes and large streams will provide sufficient oxygen for the ultimate destruction of the organic matter.

"10. Disinfection or sterilization of the sewage of a community should be required wherever there is danger of the boundary waters being so polluted that the load on any water-purification plant becomes greater than the limit above mentioned.

"11. It is our opinion that, in general, protection of public water supplies is more economically secured by water purification at the intake than by sewage purification at the sewer outlet, but that under some conditions both water purification and sewage treatment may be necessary.

"12. The bacteriological tests which have been made in large numbers under the direction of the International Joint Commission indicate that in most places the pollution of the boundary waters is such as to be a general menace to the public health should the water be used without purification as sources of public water supply or should they be used for drinking purposes by persons traveling in boats.

"13. It is our judgment that the drinking water used on vessels traversing boundary waters should not be taken indiscriminately from the waters traversed, unless subjected to adequate purification, but should be obtained preferably from safe sources of supply at the terminals.

"14. While recognizing that the direct discharge of fecal matter from boats into the boundary waters may often be without danger, yet in the interest of effective sanitary administration it is our judgment that the indiscriminate discharge of unsterilized fecal matter from vessels into the boundary waters should not be permitted.

"Yours, respectfully,

"GEORGE W. FULLER.
EARLE B. PHELPS.
GEORGE C. WHIPPLE.
W. S. LEA.
T. J. LAFRENIERE."

Mr. F. A. Dallyn, who was one of the six consulting engineers to advise the Commission, presented a minority report, in which he eliminated Paragraphs 5, 7, and 11, and made a few slight changes in some of the other paragraphs. Mr. Whipple.

Screens have also an important application in the treatment of trade wastes. In fact, they are already used to a considerable extent in factories which handle fibrous material in a wet way. In England the writer has seen screens of the drum type or the cylindrical type operated so effectively as to produce a saving of material equal in value to the cost of operating them.

The author has made allusion to the extensive development of the art of screening in Germany. This may be illustrated by the following statistics collected by Mr. Sylvester Schattschneider, of the Department of Sanitary Engineering, Harvard University, as a part of a study of the sanitary conditions of certain German rivers.

Tables 17 and 18 show the number of sewage disposal plants of various kinds above the intakes of the water-works named in Column 1. In general, these have been classified according to the final process used in the treatment. The simplest form of treatment is that mentioned in Column 2, namely, coarse screens. The efficiencies of the treatments increase in order from left to right, the best effluents being obtained by broad irrigation and intermittent sand filtration. The figures do not indicate the number of screens in use, but only the number of plants where screening is the final process.

The figures include only cities which are in Germany. Some of the more important catchment areas extend into neighboring countries, so that the total number of treatment works above a given intake may slightly exceed the figures given.

Germany has followed the general principle of some treatment for all sewage, and for each the simplest treatment that will suitably protect the waterways. It will be noticed, also, that Germany does not depend on sewage treatment to protect water supplies, but on water filtration and the use of ground-water. The writer believes that, for many sections of the United States where the waterways are large, this is the best general principle to follow. The treatment selected for any given case must be adequate for the most adverse conditions, and it should not be forgotten that in many sections of the United States the streams reach a minimum flow far below that of German rivers, and also that the summer temperatures of the water in the United States are in general higher than in Germany and England. The German rivers differ from those of England in having swifter currents. Consequently, a simpler form of sewage treatment is permissible.

During the summer of 1914, Mr. Theodore R. Kendall, a recent graduate of the School of Engineering of Harvard University, has

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TABLE 17.—SEWAGE TREATMENT WORKS ABOVE WATER-WORKS INTAKES OF GERMAN CITIES SUPPLIED WITH FILTERED SURFACE WATER.

Locations of intakes. (1)	Coarse screens. (2)	Fine screens. (3)	Plain sedimentation. (4)	Coarse screens and plain sedimentation. (5)	Kremer apparatus. (6)	Emscher tanks. (7)	Chemical precipitation. (8)	Chemical precipitation and sedimentation. (9)	Contact beds. (10)	Sprinkling filters. (11)	Contact beds and sprinkling filters. (12)	Broad irrigation. (13)	Intermittent sand filtration. (14)	Broad irrigation and intermittent sand filtration. (15)	Total number of cities and villages with sewage treatment works. (16)	Total population above intake, in thousands. (17)	Total population with sewage treatment, in thousands. (18)	Percentage of total population using sewage treatment. (19)
Altona	4	1	18	3	3	6	4	2	4	3	1	12	2	0	67	22 680	6 844	28.0
Berlin (Müggel)	0	0	0	0	0	0	0	0	0	0	1	0	1	0	3	800	105	13.0
Berlin (Tegel)	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	295	19	4.5
Braunschweig	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	90	95	25.0
Bremen	3	1	3	4	0	0	0	0	5	3	1	4	1	0	38	4 138	1 997	47.0
Breslau	0	0	3	0	0	0	0	0	4	3	0	3	1	0	16	3 345	240	7.0
Brieg	0	0	3	0	0	0	0	0	0	0	0	2	0	0	5	310	208	9.0
Frankfurt	4	1	6	3	3	6	4	2	4	7	1	8	3	0	33	6 496	1 094	17.0
Gütersloh	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	23 023	6 350	28.0
Güstrow	3	1	18	3	3	6	4	2	4	7	1	8	3	0	66	16	0	0
Hamburg	3	1	18	3	3	6	4	2	4	7	1	8	3	0	66	16	0	0
Hannover	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	545	412	25.0
Königsberg	3	1	3	1	3	0	0	0	4	1	1	1	0	0	3	412	54	12.0
Lübeck	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	434	0	0
Magdeburg	2	1	10	2	0	3	0	0	3	0	0	3	0	0	21	0	0	0
Posen	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	930	0	0
Ratibor	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	880	247	14.0
Rostock	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	695	25	1.0
Schwerin	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	840	0	0
Stralsund	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	91	18	20.0
Strassfurt	2	0	0	1	0	0	0	0	0	0	0	0	0	0	3	2	0	0
Stettin	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
Stuttgart	1	0	3	0	0	0	0	0	0	0	0	0	0	0	5	289	118	47.0
Wandsbeck	3	0	9	0	0	0	0	0	0	0	0	0	0	0	44	651	484	12.0
Worms	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	459	4	0.9
Zürich	2	0	4	0	0	0	1	0	0	0	0	0	0	0	20	8 090	871	11.0
	32	6	106	20	13	28	9	4	24	24	5	41	11	0	348	98	.....	.....

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TABLE 18.—SEWAGE FILTRATION STATISTICS OF GERMAN CITIES OF 10 000 OR MORE INHABITANTS ABOVE DIFFERENT INTAKES.

Locations of intakes. (1)	Number of cities above intake. (2)	Cities without treatment. (3)	Coarse screens. (4)	Fine screens. (5)	Plain sedimentation. (6)	Coarse screens and plain sedimentation. (7)	Kremer apparatus. (8)	Emscher tanks. (9)	Kremer apparatus and Emscher tanks. (10)	Chemical precipitation. (11)	Chemical precipitation and sedimentation. (12)	Contact beds. (13)	Kremer apparatus and contact beds. (14)	Sprinkling filters. (15)	Contact beds and sprinkling filters. (16)	Broad irrigation. (17)	Intermittent sand filtration. (18)	Broad irrigation and intermittent sand filtration. (19)	Foreign cities on German catchment areas. (Information not available.) (20)
Altona	113	41	8	1	9	1	3	3	0	8	2	3	0	5	1	10	2	0	28
Berlin (Müggel)	4	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Berlin (Tegel)	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Braunschweig	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bremen	24	9	1	1	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Breslau	15	7	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Brieg	15	8	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Frankfurt	92	16	3	1	3	1	0	3	0	0	0	0	0	0	0	0	0	0	0
Glückstadt	115	43	3	0	3	0	0	3	0	0	0	0	0	0	0	0	0	0	0
Güstrow	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hamburg	111	40	3	1	3	1	0	3	0	0	0	0	0	0	0	0	0	0	0
Hannover	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Königsberg	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Lubeck	85	33	3	0	3	0	0	3	0	0	0	0	0	0	0	0	0	0	0
Magdeburg	6	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Posen	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ratibor	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Rostock	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Schwerin	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Stralsund	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Stassfurt	4	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Stettin	44	17	3	1	3	1	0	3	0	0	0	0	0	0	0	0	0	0	0
Stuttgart	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Wandsbeck	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Worms	40	19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Zürich	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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visited many of the sewage disposal plants of Europe, and the writer takes pleasure in presenting the following notes on visits which he made to places not mentioned by the author.

Eberswalde.—At Eberswalde two sets of screens precede the settling and biological treatment, the first set consists of round bars 2 cm. in diameter and with 3-cm. spacings. This arrests the coarser material and paper, and is cleaned by hand when necessary. Immediately beyond is a Reinsch-Wurl screen with 3-mm. openings. All screenings are elevated to the surface and discharged into metal cans holding about ⅓ cu. yd. each. The screenings sell for about 2.5 cents per can. There was no odor about this plant when the temperature was 35° cent. (95° Fahr.). Stadtbaurat Arndt mentioned that before screens were installed the sedimentation basins had to be cleaned every 3 weeks; now they can be used from 4 to 5 months. Population, 37 000.

Düsseldorf.—The sewage of Düsseldorf is approximately 1 cu. m. per sec., giving a velocity of 0.78 m. per sec. in the main outfall. The flow passes a *grobrechen* or coarse screen with six wings, one of which is in the sewage; when this becomes slightly clogged, an attendant turns it and removes the screenings. These wings are about 3 m. long. The fine screens are of 2-mm. piano wire with 2-mm. spacing, and are on large steel frames. There are six of these screens, three of which are sufficient for ordinary flows. The fine screens are cleaned by rotating arms which rake off all screenings, and are in turn cleaned by a scraper which removes the screenings to a continuous conveyor. The average quantity of screenings is 13 cu. m. per day (10 cu. m. between 8 A. M. and 9 P. M., and 3 cu. m. from 9 P. M. to 8 A. M.). The plant has been in operation 10 years, and costs about 40 000 Marks ($10 000) per year for operation and repairs. The screened sewage contains only ½% of sediment after standing 2 hours. To operate all screens requires only 6 h. p., which is furnished by a steam engine capable of producing 15 h. p. The screenings are piled in large concrete bins having under-drains, and are covered with *torf* and chalk to keep the odor at a minimum. The screenings sell in October for 1.50 Marks per 1 000 kg. (36 cents per ton). Population, 400 000. Sewered area, 25 000 acres.

Cologne.—The sewage is screened by 0.5 and 2.5-mm. screens, and these are cleaned by wire brushes which carry the screenings to an endless conveyor and thence to a car which removes them to concrete pits for storing, as at Düsseldorf. The screenings sell at 0.50 Mark (12 cents) for a 2-cu. m. wagon load, and 0.30 Mark (7 cents) for a 1-cu. m. load. The odor here was nil at the screens, but very bad at the compost piles. The screens remove about 15 cu. m. per day from 75 000 cu. m. of sewage. Population, 440 000. Future works are to be built with Reinsch-Wurl screens because of the greater accessibility of the whole screen and cleaning devices for repairs.

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Wimbledon, England.—The screens consist of fixed bars, with engaging teeth to clean them, the latter being operated by motors. Similar screens are in use at Chester, Birmingham, Salford, Wakefield, and Huddersfield, at present, and differ widely from the Carshalton screen shown in this paper, inasmuch as the screen is fixed and the cleaning device movable, whereas the Carshalton screen itself moves and is cleaned at the top.

Rochdale, England.—The same general type of screen is in use in Rochdale as at Wimbledon, but the method of cleaning is slightly different and worthy of mention.

"The three screening and raking apparatus (Law's patent) are fixed in chambers 8 ft. wide and 6 ft. 9½ in. deep at the outlet end * * *. Each inclined screen is fitted with taper section steel bars 7 ft. 6 in. long, spaced ½ in. apart, the maximum sewage level being about 5 ft. The raking gear above the coping consists in each apparatus of a built-up steel framework * * * carrying a travelling-cleaning rake fitted with * * * malleable prongs. The rake is suspended by two vertical 'T'-section arms from a cross-bar on the chains, and is counterbalanced * * *. The ends of the rake are fitted with runner pulleys, working between cast-iron guides in front of the inclined screen. In working, the rake descends through the guides farthest from the screen, and ascends through the guides nearest to the screen, the prongs engaging in the spaces between the screen bars and removing the refuse therefrom. A vertical steel plate is fitted at the top of the screen to prevent the refuse falling from the rake, and, on the rake arriving at the highest point of travel, the refuse is automatically discharged by a cleaner, consisting of a set of vertical prongs in a cross-bearer, actuated from the detachable chains which carry the rake. At the junction of the 'up' and 'down' guides a hinged flap is fitted, to ensure the rake descending the guides farthest from the inclined screen."*

North Bierley, near Bradford, England.—Wool fiber and picric acid are the chief trade wastes in this locality, so a screen was designed to be as completely accessible as possible and still effectively remove the fibers. The screen consists of a perforated sheet of copper, about 7 ft. square and bent to form a half cylinder, along which the sewage flows. Revolving wire brushes pick up the lint and other solids and are in turn cleaned by a straight bar which passes over them when they reach the upper edge of the screen. This type of screen was adapted from the screens used in the mills near-by.

Rothwell Urban District Works, near Wakefield.—The use of a rapidly revolving disk screen at this plant, which receives a daily dry-weather flow of 54 000 gal. from 4 000 people, is chiefly to break up the solid matter before sending it to the trickling filter. This is probably the only plant of this type in existence. The sewage first

* Institute of Municipal and County Engrs., June 14th, 1913, by the courtesy of Mr. S. S. Platt, Borough Surveyor.

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passes a bar screen with ½-in. openings, and then is pumped up to the disk screen, which revolves rapidly in a horizontal plane, allowing the liquid and most solids which are well broken up to pass through the ¼-in. holes and rejecting practically only mineral and other heavy solids which are not easily broken up. These are caught in a tray at one side and removed to be buried, as the volume is very small. It may be well to add that the trickling filter bed is chiefly composed of broken brick with a 2-in. top covering of 1-in. gravel.

Colehall, near Birmingham.—This is a rather good example of a trickling filter installation working well without screens. This plant was constructed some years ago, but within a year has come under the control of the Birmingham Corporation. The sewage is practically identical with that of Birmingham. The population served is about 60 000, and the flow is 2 400 000 gal. daily. The flow is directed through three settling basins in series, the heaviest material, chiefly mineral, settling out in the first and the finer in the latter basins. The basins are cleaned by a clam-shell bucket operated from a track at the side. The first basin is cleaned every 2 weeks, the second every 4 weeks and the third every 8 weeks. If a heavy storm occurs when the first basin needs cleaning, the material is simply flushed over into the next basin. At no time has there been any trouble with detritus being washed past the third basin. The results of the operation are: Suspended matter, raw sewage, 500 to 800 parts per million; influent to sprinklers, 50 to 120 parts per million; oxygen absorbed, raw sewage, 200 to 300 parts per million; influent to sprinklers, 50 to 90 parts per million.

The author does well to emphasize the need of greater system in keeping the results of operation of sewage screens and of weighting the results of samples on the basis of the corresponding volumes of sewage flow when the samples were collected. He might have added, also, the need of more frequent sampling.

In spite of the greatest care in devising and maintaining systems of sample collection and analysis, the results will be understood by only a few. There is need of a simpler and more tangible record of suspended matter than that which is used.

An excellent method of keeping such a record is that of filtering samples of water through cotton disks. A given volume of water, say 1 gal., is filtered through a disk of absorbent cotton, about 1 in. in diameter, specially prepared for the purpose. This is most conveniently done with a simple apparatus known as the "Wizard Sediment Tester" which is made by the Creamery Package Manufacturing Company, of Albany. The cotton disk, which is the filtering medium, is held between two supports of wire cloth in a cap attached to an ordinary glass milk bottle. The water to be filtered is placed in the bottle and allowed to flow out through the cotton disk. Filtration is hastened by increasing the pressure of the air within the bottle by the use of a simple air

compressor operated by a hand bulb. For samples of sewage, a much smaller quantity than 1 gal. is required. The cotton disk sediment record has the following advantages: Mr. Whipple.

1. It can be made very easily and quickly;
2. It can be made by the laborer and the office boy as well as by the engineer or chemist;
3. It is inexpensive;
4. The records are picturesque, and easily understood by every one;
5. The records are permanent;
6. The records may be conveniently mounted for preservation;
7. The records may be photographed;
8. A relatively large volume of water is tested; and
9. It is a valuable supplement to the regular water analysis.

C. A. JENNINGS,* Assoc. Am. Soc. C. E. (by letter).—Mr. Allen has compiled a mass of useful and interesting information and data on the subject of fine-screening of sewage. The writer has had much difficulty, in searching English publications, in finding very much reliable data on this subject. This paper will serve a very useful purpose. There is no doubt that in the United States fine screens have not received the attention they deserve. Whereas numerous experiments have been made on the various types of settling tanks, none has been made on the use of fine screens, so far as the writer is aware. Mr. Jennings.

The Jennings screen was mentioned by Mr. Allen as a recently patented type of band screen. This screen is handling successfully in Chicago a typical stock-yards sewage, which is rich in corn, straw, and hay, and cattle, hog, and sheep manure, and there is also present some human sewage. It comes from an area of some 30 acres, on which are live-stock pens, streets, buildings, etc. Formerly, the screen was cleansed by air under a pressure of about 45 lb., delivered through $\frac{3}{64}$-in. perforations in a $1\frac{1}{2}$-in. pipe. These holes became clogged with rust from the air pipe, and the high pressure of the air wore out the screens rapidly. This has been entirely remedied by supplying about 500 cu. ft. of air per min., under a pressure of $1\frac{1}{2}$ lb. per sq. in., through a slotted 3-in. pipe. The air is supplied by a positive blower driven by the same motor that drives the screen. This has overcome the objections to the high-pressure air, and, at the same time, has increased the efficiency of the screen. The data in Table 19 are from a forthcoming report of the Sanitary District of Chicago, dealing with the sewage problem at the Chicago Stock Yards. The writer is indebted to George M. Wisner, M. Am. Soc. C. E., Chief Engineer, and Langdon Pearse, M. Am. Soc. C. E., Division Engineer, for their use, the data having been collected under their direction.

* Chicago, Ill.

Mr. Jennings.

TABLE 19.—CRUDE SEWAGE.

Test No.	Rate of flow of sewage.	Gallons per square foot of screen exposed.	MOIST SCREENINGS.		DRY SCREENINGS.
			Pounds, per million gallons.	Percentage of moisture.	Pounds, per million gallons.
1.....	1 120 000	2.11	6 170	83.0	1 050
2.....	1 138 000	2.14	4 820	80.4	945
3.....	1 440 000	2.71	7 860	82.0	1 420
4.....	1 305 000	2.45	7 390	86.6	990
5.....	1 203 000	2.37	7 200	82.5	1 260
Average...	1 241 000	2.36	6 690	82.9	1 130

NOTE.—With screenings weighing 55 lb. per cu. ft., the volume of screenings varied from 3.2 to 5.3 cu. yd., and averaged 4.5 cu. yd. per million gallons of sewage screened.

Mr. Allen mentions the high moisture content of the screenings. This was due, at that time, and in the data in Table 19, to the method of constructing the screen frames of this first installation. Due to particles of corn, coal, gravel, etc., rolling down the screen, a 2 by 2 by $\frac{1}{4}$-in. angle-iron, with an outstanding leg pointing up on the lower side, was substituted for the $\frac{3}{4}$ by $\frac{1}{8}$-in. plate. This angle-iron retained the larger particles which formerly rolled down the screen, but it also retained considerable water until the frame reached the top of the incline where the blast of air blew off the screenings. This trouble has now been overcome by perforating the angle-iron with $\frac{1}{2}$-in. holes, so that the water drains out before the screen reaches the top of the incline. A number of tests made recently showed the average moisture content of the screenings to be 73.5%, as against a former average of 79 per cent. These wet screenings are easily handled with a shovel or pitchfork. They are dark green in color, and, while fresh, the odor is inoffensive. Table 20 gives typical analyses of the screenings.

TABLE 20.

Percentage of moisture.	CALCULATED TO DRY WEIGHT—PERCENTAGE.			
	Nitrogen.	Volatile matter.	Fixed matter.	Ether, soluble.
83.0	2.24	93.0	7.0	4.1
80.4	1.60	94.0	6.0	3.0

NOTE.—These data are from a forthcoming report of the Sanitary District of Chicago on the Stock Yards sewage problem.

On a recent installation, the percentage of moisture in the screenings has been reduced by reversing the angle-iron referred to, so that

Mr. Jennings.

it carries no water with it, the water draining through the screen immediately.

The writer is of the opinion that percentage-removal data mean little. Probably the technique of this determination differs in every laboratory. When one considers the immense quantity of sewage screened, the very small portion of this volume going to make up the composite sample, the small quantity of the composite sample used for the determination, and finally the size of the particles in suspension in the raw sewage, it is easy to see many chances for error in making this determination. The writer has seen a chemist use a 10-cu. cm. pipette to collect a sample of raw sewage to determine the suspended matter, in spite of the fact that very few of the particles in suspension could be sucked up through its tip. Not less than 40 or 50 cu. cm. should be used in making this test, and the sample should be collected in a graduate, in the top of which a funnel is placed. The bottle of sewage is shaken in an inverted position above the funnel and graduate, and the stopper or one's hand quickly withdrawn and sufficient sewage allowed to enter the graduate through the funnel. This method has been tested by the writer and found to give comparable results.

Sewage from about thirty-five packing houses enters Bubbly Creek, or the Stock Yards Slip, a branch of the Drainage Canal. This sewage is quite strong. It contains considerable fat and particles of skin and hair which are not intercepted in the catch-basins of the various packing houses. The Jennings screen is now handling a similar sewage in another city. At Chicago, this packing-house sewage receives no treatment other than the passage through catch-basins. It is with this problem that the Sanitary District of Chicago has experimented for about two years, and a report is now being prepared on the results of this work. It seems to the writer that here is an excellent field for fine-screening. The water from the Drainage Canal is not used for domestic consumption, and, furthermore, the particles that can be removed by fine screens are those which would settle to the bottom of the canal to a great extent and form deposits. In this way much of the small quantity of dissolved oxygen in the canal would be used up. It is desirable to remove this matter in suspension, and a cheap, effective, and quick method of doing so is by fine-screening.

Other fields for fine screens have appeared in manufacturing plants where the problem is either to recover something in suspension which has value, or to prevent something in suspension, which has no value, from being discharged into a stream or other body of water. The more the writer has gone into this subject, the more he is convinced that fine-screening of sewage has been overlooked to a large extent, and that there are many places where its application is as good as, or, in some instances, even preferable to, tank treatment.

Messrs. Franze and Schaefer.

MESSRS. G. FRANZE* AND — SCHAEFER† (by letter).—The author's painstaking work forms an important contribution in settling the question of the importance of screening installations in the field of sewage clarification. While acknowledging the correctness of Mr. Allen's deductions and conclusions, the writers beg to present some additional observations founded on their experience.

In considering a plant about to be installed, one should provide for the most complete treatment that may be demanded, if he wishes to satisfy the variety of requirements that may arise eventually for the cleanliness of the stream. A plant, therefore, should be planned and provided from the beginning with everything which later may be necessary for the kind of treatment which has proved satisfactory in all cases. Its development in accordance with the more complete demands of the future can then be temporarily postponed, provided the conditions are such as to make this advisable. In such cases, the first installation of the plant should provide for mechanical cleansing by screens (bars, sieves, grids, etc.).

This procedure is to be preferred under the following circumstances:

(1).—As an independent and separate arrangement for the removal of suspended matter greater than a given size where the stream to which the effluent passes is large, thereby providing ample dilution, and where there are no communities below which require the use of the water; and also where there are æsthetic reasons for preventing the fouling of the shores.

Such independent arrangements, however, have no influence on the removal of dissolved impurities. These are disposed of by the biological purifying agencies contained in the stream itself, as long as the volume of sewage remains within certain limits relative to the self-cleansing ability of the stream.

(2).—As a preliminary clarification plant in connection with other processes for the further removal of the finest suspended matter. In this case screens form also an independent and necessary feature:

(*a*) During periods of flood in the stream;

(*b*) Without affecting unfavorably further clarification;

(*c*) Without preventing the subsequent treatment of the sludge by machinery.

(3).—As a protection to pumps in systems of low-level sewers.

The conditions affecting plants mentioned under (1), (2), and (3), depend further on the kind and composition of the domestic and factory wastes. The best and most efficient results are with fresh sewage. The most unfavorable are with stale sewage, where the im-

* Stadtbaurat, Frankfort, Germany.

† Stadtbaumeister, Frankfort, Germany.

purities reach the plant in a finely broken up and decomposed condition. The most important advantages in mechanical treatment are the following: Messrs. Franze and Schaefer.

(1).—The product contains only about 75% of moisture and is, therefore, in its firm condition, capable of transportation. The quantity removed, if carried to tanks or other arrangements, would increase the quantity of sludge by from 10 to 15 per cent.

(2).—The fertilizing value of the material is greater, thus paying for its carriage to a greater distance.

(3).—The removal of coarse and clogging material renders possible an unobjectionable and certain removal of sludge from the tanks which, in some cases, must be provided with complicated pipe systems for its removal either by suction or pressure.

(4).—This also holds good in the further mechanical treatment of the sludge.

(5).—The simple and automatic machinery requires only a small sum for service and power.

With regard to the agricultural use of material removed by mechanical devices, it was formerly given to neighboring farmers. After experiments made at Frankfort, Germany, however, it is now possible to dry the product by the chimney gases of the refuse incinerator so completely, in a drying drum, that the heat developed in burning can be utilized for electric power, or the material can be used on farm lands as poudrette.

The choice between the successful mechanical plants now known—which may be divided into two groups, fixed and movable—should be made in each case only after a thorough study of local conditions as well as of the character of the sewage and the stream into which it is discharged.

GEORGE W. FULLER,* M. AM. SOC. C. E. (by letter).—Mr. Allen has written a good paper on a subject about which practical knowledge in the United States falls far behind that which is available in Europe. Undoubtedly, his conclusion is correct, that fine screens for sewage clarification are entitled to more consideration than they have generally received hitherto in America. Mr. Fuller.

As a general proposition, it may be safely stated that fine screens have their legitimate field of usefulness where the clarification effected by them can be accomplished at less cost than by other available methods, when due consideration is given to capital charges and depreciation, as well as to ordinary operating expenses and current maintenance. In some places where the total cost of screens might be somewhat greater than that of settling tanks, the convenience may justify their use.

* New York City.

Mr. Fuller.

Fine screens are most serviceable in cases where clarification alone or in combination with sterilization is sufficient treatment to give to the sewage. Where settling tanks are used, fine screens do not seem to be of sufficient aid to justify their cost. Perhaps exceptions may be found in instances where some by-products of value are to be found.

In connection with filters of various types, fine screens do not seem to have established themselves as of much practical aid if exception be made of cases like Baltimore, where gas ebullition lifts much sludge at times from the single-story tanks on to the filters. The experience of Brockton is of much interest, and it would be helpful to know how expenditures for fine screens there would compare with what could be accomplished by modern settling tanks, in lessening the load on the sand filters.

Last spring, the writer designed a sewage treatment plant at Daytona, Fla., in which Riensch-Wurl screens (with a width of opening of $\frac{1}{12}$ in.) and sterilization afford the only treatment before the dispersion of the sewage in the Halifax River not far distant from shellfish layings. It was concluded that these fine screens would give sufficient clarification and that the cost would be materially less than for settling tanks for a city where quicksand underlies practically the whole area, with its upper surface from 6 to 12 ft. below the surface of the ground and extending to depths of some 30 to 40 ft.

Mr. Allen's paper has much value for reference, on account of the European data which it presents. The writer believes, however, that actual experience with fine screens in the United States is likely to show a smaller removal of suspended matter than would be inferred from statements in the paper. Thus, at Reading, Pa., there is quoted a removal of dry matter equal to 90 parts per million, whereas, for several years, the writer's studies indicated that, on a basis of daily averages, this removal rarely if ever exceeded 30 parts per million. Without doubt, the comparatively dilute American sewages cannot be clarified with the same relative efficiency by fine screens as the concentrated sewages of Europe. As the sewage becomes comminuted, through age and mechanical attrition, incident to flowing through long outfall sewers, the efficiency of fine screens will become less.

Generally speaking, fine screens have not been built strongly enough, either in the United States or in the earlier installations in Europe, to do their work satisfactorily without more or less overhauling and reinforcing. Without such heavier construction, the periods of interruption and the cost of attendance become relatively great. There should be due allowance for this factor in making a preliminary estimate of cost.

Notwithstanding the foregoing comments as to lessened efficiency, as compared with the author's statements, and the likelihood of greater cost, the writer believes firmly that in the future fine screens will play

a prominent part in solving many problems of sewage treatment in America. Mr. Fuller.

J. X. COHEN,* JUN. AM. SOC. C. E. (by letter).—The author deserves credit for the presentation of data (undoubtedly collected with much effort) which enrich the meager American literature on sewage screening. He has brought to the attention of American engineers a process of sewage treatment which merits more attention than it has received as yet in the United States. Mr. Cohen.

Most sewage screens in the United States are of the crude bar type, and although they serve their intended purpose more or less effectively, there are without doubt a number of plants where improved screens, if planned with the painstaking care shown in most of the designs presented in the paper, would give better results.

The success of the mechanical screens discussed by the author, however, will become possible in America only when more of our municipalities come to hold enlightened views regarding the necessity for skillful operation and scientific supervision in sewage treatment works. It will be of little avail to install an expensive screen of high efficiency, and then allow its control to be vested in an operator of the type so usual in numerous American sewage disposal works. Whereas the methods and structures now in use are capable of more abuse in practice, without great deterioration in the quality of the effluent, than are mechanical screens, and in consideration of the present attitude of many public officials, that sewage disposal works are necessary evils, frequently forced on them by Court orders, which must be tolerated, but to which as little attention as possible should be given, sanitary engineers who realize the conditions fully, will hesitate to recommend the installation of automatic screening machinery.

On the other hand, it may be that the installation of such screens in progressive communities, where problems of municipal management of public works are being solved, not from the viewpoint of political exigency, but from that of scientific reliability, will present concrete object lessons in economical municipal management that in time will be learned. Without this recognition of the importance and the vital necessity for reliable sewage works operation, however, much of the effort and expense involved in the design and installation of high-efficiency, sewage-screening machinery will be lost.

The writer is of the opinion that, in the design of sewage screens, the convenience of the attendant has not always been kept in mind. This opinion is based on an experience of about 8 years, and refers in particular to small plants. The result of this lack of prevision is that screens are either improperly cared for, neglected, or are completely removed from the sewage flow.

* New York City.

Mr. Cohen.

As it is a well-known fact that many small sewage disposal works are poorly conducted and therefore produce effluents which are not wholly satisfactory, it should be the duty of the designing engineer to arrange the various parts of the plant so that in their operation the attendant, at all times, will be as free as possible from arduous labor in awkward positions. As an instance of the apparent failure to keep in mind during design the practical operation of a screen, there may be mentioned a small disposal works near New York City where the operator has found the task of cleaning the screen so difficult, because of its position, that it has been neglected for a long time, with the natural result that it clogged and the sewage on its way to the pumps overflowed the screen. Thus far, it is not known that this has caused any trouble at the pumps, but, of course, deposits have accumulated above the screen, with the accompanying nuisance. As this particular screen-chamber is covered, it will be some time, perhaps, before the odors of decomposition become sufficiently obnoxious to force the removal of the deposits.

In another small plant, the basket-bar screen is too heavy and cumbersome for the lone operator to handle conveniently, and the result has been the withdrawal of the screen from the well. The sewage now flows unscreened and uninterrupted to the pumps, thereby endangering their safe and continuous operation. Such practices do not enhance the respect of the operator for the engineer, nor redound to the credit of the latter.

There is a feature of fine sewage screening that will be of service to its development in America, which deserves considerable attention. Centrifugal pumps are now used extensively in pumping sewage, and in most recent designs other types have not been given equal consideration. Fine screens will eliminate the grosser solids found in coarse screened sewage, and the manufacturers of centrifugal pumps will be enabled to design impellers and pump casings which will allow the pumps to attain higher efficiencies than are now feasible. For large flows, the saving in the cost of pumping thus obtained will be a factor which should have considerable and favorable influence on the decision concerning the use of mechanical screens.

Mechanically operated fine screens in large plants probably receive the skilled attention which they require, but in smaller plants, where the quality of the effluent may be equally important, if not more so, such screens may not always receive as good attention. Hence, it behooves engineers to give sufficient study to the design of sewage screens, whether of the simple stationary bar type or the more complex mechanical type, in order that, under practical operating conditions, they may receive more than sporadic attention. Such attention, or rather lack of attention, forces the other parts of the disposal works to exercise functions for which they are not designed, with the con-

sequent lowering of their efficiency. By presenting to the Society data which indicate how other engineers are ingeniously working out the problems of sewage screening, the author has made a noteworthy contribution which should stimulate other members to equal endeavors. Mr. Cohen.

F. T. ROBSON,* Assoc. M. AM. SOC. C. E. (by letter).—While engaged in the design of a sewage disposal system in Southern California, the writer heard of the great difficulty that The Anaheim Sugar Company was having in disposing of its waste water which carried a large percentage of sugar-beet waste. The factory was in a thickly settled, highly developed farming district containing many small cities, and at such a distance from the ocean as to make it an economic impossibility to discharge the waste water there. It might have been delivered directly to the citrus orchards immediately adjoining, or into a ditch for irrigating lands at a greater distance. Most of the farmers refused to use the water, as the factory operates for the entire 24 hours of the day during a "campaign", and night irrigation, as well as painstaking attention to cover or plow under the large quantity of suspended matter, would thus be necessary. Mr. Robson.

The factory had been completed and put in operation in 1911, and the campaign of 1913 was fast approaching when the writer was asked to report on the problem. In 1911 the wastes were discharged in an artificial 55-acre lagoon near the plant, and readily decomposed under anærobic action, producing much hydrogen sulphide. Many serious complaints from near-by residents resulted. In 1912 the wastes were discharged into an irrigation ditch, but the large quantity of organic suspended matter caused difficulty similar to that in the previous year. Complaint was made to the State Board of Health, so, in 1913, the company was compelled to abate the nuisance or close the factory. It was essential that something be done at once, for the time was short.

In local parlance the word "campaign", used in connection with a sugar factory, means the period from the time when the first beets are received, or the factory is started, until the last beet has been cut up and the factory closed. During this time the factory runs 24 hours per day and a shutdown is a serious matter. All energies during the "closed" period are devoted to putting everything in such shape as to minimize the chance of a breakdown.

After careful study the only available methods of solving the problem seemed to be:

(1) Fine screening;
(2) Sedimentation.

The dearth of information regarding recent installations of fine-screening devices, their cost, operating difficulties, etc., was astonishing.

*San Francisco, Cal.

Mr. Robson.

For this reason Mr. Allen's paper is especially valuable, and should be of wide interest, for fine screening is available as a method of sewage disposal in a large number of cases. He has collected in one paper far more information, as to design and operation, than was found by the writer in a number of authorities consulted.

On investigating this local matter, many aspects differing from the usual sewage disposal problem were found, and Charles Gilman Hyde, M. Am. Soc. C. E., Professor of Sanitary Engineering, University of California, was asked to assist. His report was very detailed and comprehensive, and of great value, because it was the first time in California and probably in America that the disposal of beet sugar factory wastes had been studied scientifically. Tables 21, 22, etc., are taken from Mr. Hyde's report to the writer.

TABLE 21.—CHEMICAL ANALYSIS OF ROUGHLY SCREENED BEET SUGAR WASTE.

	Determinations.	Parts per Million.
Solid matters	Total	2568.0
	Mineral	1521.6
	Organic and volatile	1066.4
Suspended matters	Total	1891.2
	Mineral	1208.0
	Organic and volatile	683.2
Dissolved matters	Total	696.8
	Mineral	313.6
	Organic and volatile	383.2
Nitrogen (N) as	Organic nitrogen	7.14
	Free ammonia	5.60
	Albuminoid ammonia	1.80
	Nitrites	0.19
	Nitrates	0.22
	Total	13.15
Oxygen consumed (O)	"Absolute"	1985.
	5 min. at 100° cent.	496.
	20 min. at 80° cent.	198.50
Sulphates (SO_3)	Total (dissolved)	50.60
Chlorine (Cl)	Total (dissolved)	40.00
Phosphoric acid (P_2O_5)	Total (dissolved)	Trace.
Potash (K_2O)	Total (dissolved)	10.05
Soda (Na_2O)	Total (dissolved)	43.50

As far as known, this installation is the first in which sugar factory waste has been treated by fine screening; it is also the first installation of a screening device of any kind in California. A brief description may be of interest.

Investigations preliminary to design showed that the volume of waste requiring treatment was about 2 200 000 gal. daily during the campaign (approximately from August 1st to November 1st).

Mr. Robson.

General Character of Waste.—The waste consisted of discolored water heavily loaded with suspended matter most of which was heavier than water—some lighter; it was largely sand, silt, clay, beet tops, roots, cuttings, etc., the impurities from beets or unpurified sugar, and the sewage from 200 employees.

Chemical Analysis of Waste.—The result of a chemical analysis of roughly screened waste, made at the University of California, is given in Table 21; and from this it follows that the suspended solids were equivalent to about 15 800 lb. per million gallons.

Analysis of Water.—The mineral composition of the water carrying the waste is shown by Table 22.

TABLE 22.—ANALYSIS OF WATER CARRYING WASTE.

Determinations.		Parts per Million.
Total solid matters	Total	397.0
	Mineral	357.0
	Organic and volatile	40.0
Suspended matters	Total	0.0
Dissolved matters	Total	397.0
	Mineral	357.0
	Organic and volatile	40.0
Sulphates (SO_4)	Total (dissolved)	22.2
Chlorine (Cl)	Total (dissolved)	34.5
Alkalinity due to sodium carbonate		6.4
Incrusting solids:		
Calcium and magnesium carbonate, etc., chiefly small		244.0
Calcium sulphate (Gypsum)		
Silica (SiO_2)		10.0

Volume of Suspended Matter.—(*1*) *By Imhoff Graduates.*—

Time of subsidence.	Average volume.
1 min	6.3 cu. cm.
2 "	6.9 "
3 "	7.1 "
4 "	7.2 "
5 "	7.3 "
10 "	7.4 "
15 "	7.6 "

This is equivalent to 37.8 cu. yd. per million gallons, or, for the daily flow of 2 200 000 gal., a total of 83.2 cu. yd. of wet sludge having a moisture content of about 95%, giving a total dry weight of 3 170 lb. per million gallons, or 34.87 tons per day after 15 min. sedimentation. Computing the total suspended solids as 15 800 lb. per million gallons, the quantity capable of settling in 15 min. would be about 20 per cent.

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	Mineral	313.6
	Organic and volatile	363.2
Nitrogen (N) as	Organic nitrogen	7.14
	Free ammonia	5.60
	Albuminoid ammonia	1.80
	Nitrites	0.19
	Nitrates	0.22
	Total	13.15
Oxygen consumed (O)	"Absolute"	1985.
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Mr. Robson.

Volume of Suspended Matter.—(2) By Screening.—Brass sand-analysis sieves, of 40 and 100 mesh, in tandem, gave the following results:

On the 40-mesh screen, from 17 240 cu. cm. of waste, 71.3 grammes wet, or 3.7 grammes dry, were caught; or, on the basis of 15 800 lb. per million gallons, the percentage was 11.3%, or an estimated quantity of 46.6 cu. yd. of wet material per day.

On the 100-mesh screen, 11.4 grammes wet, or 0.9 grammes dry, were retained; or, on the basis previously given, about 14.0% was retained with both screens.

Although these percentages appear to be low, the material removed represents that portion of the waste matter that caused the trouble by decomposition.

Location of Plant.—The factory is on an old river bed or channel of sand, which is very fine in the top layers, but grows coarser with depth. Tests showed the perfect feasibility of intermittent filtration, after the top layers of sand were removed. It was thus determined that some process of preliminary treatment followed by sand filtration must be used. The following figures give an idea of the estimated cost of the complete plant of the two types best adapted to the work:

	Cost of construction.	Annual operating expenses.
Fine screening and intermittent sand filtration	$32 500	$6 600
Sedimentation in Imhoff tanks and intermittent sand filtration	84 500	8 900

These annual operating costs include interest on the investment at 6%, depreciation, maintenance, labor, and power costs.

The fine-screening method, with a battery of two screens, having been recommended, plans were prepared. The company, however, expressed some doubts as to the success of the plant, and decided to invest in only one screen for the present, though space was provided in the screen-house for two.

A Weand rotary screen of the Atlanta type, furnished by the Merritt Hydraulics Company, was put in. It is cylindrical, 10 ft. long, 8 ft. in diameter at the inlet and 6 ft. at the outlet, and is covered with a 40-mesh screen. It is chain-driven from a 5-h.p. motor at a speed of 10 rev. per min. The lower half of the inlet end is closed, but there is an opening for the supply pipe. The water entering the screen filters through the bottom portion, and the suspended matter is carried by worm plates to the outlet and deposited by a chute in an iron push-car with a perforated bottom. The screen is kept clean by water jets or nozzles.

Mr. Robson.

The screenings are dumped on an open compost heap, where they gradually dry, with no disagreeable odor. When dry, the compost is used by the farmers as a fertilizer.

The screened water passes from the screen to a sump and thence by gravity to thirty-six filter beds, each 100 by 200 ft., or a gross area of 26 acres. These beds are covered once in 24 hours with about 4 in. of the screened water. They perform their function satisfactorily, and no odor is apparent. The plant is now going through the second campaign, and the results have been very successful.

The screen itself, however, has given a great deal of trouble. It was doubtless the first one manufactured by the present makers and, at the time of purchase, they had not fully completed their arrangements with Mr. Weand's heirs. The mechanical details of the screen were very poor, and for this reason the operating delays were numerous. In fact, the screen was almost completely rebuilt during the season. Notwithstanding annoyances and delays of this nature, the company has been well satisfied, and the total capital expense for the complete plant, to date, has been about $30 000.

The cheapest alternate method of disposal would doubtless have cost three times as much, could not have been finished in time for the 1912 season, and would have been more expensive to operate.

The wet sludge or screened material has been found to contain about 92% of moisture and to weigh about 45 lb. per cu. ft., as it comes from the screen, without compacting. The quantity of sludge varies largely with local conditions in the factory, but the volume per day or week checks closely with that estimated.

Mr. Potter.

ALEXANDER POTTER,* ASSOC. M. AM. SOC. C. E.—The speaker would like to add a tribute to the thoroughness with which the author has presented this subject. Although experience with fine sewage screens in the United States is limited, this is not true of Europe where fine screening is in more general use. This is especially true in Germany, where, since 1903, the Government has sanctioned the discharge, into large bodies of water, of sewage from which all suspended matter more than 3 mm. in diameter has been removed.

The principal contributing causes for the relatively small use of fine screens in America, as compared with their use in Europe, appear to the speaker to be the following:

First.—The failure of competent authority to establish reasonable regulations as to what would constitute adequate treatment under all conditions. There has been too great a tendency to permit the gross pollution of waters by sewage for much longer periods than are desirable, and these periods have been lengthened unconscionably by the insistence of too many of the State Boards of Health on standards

* New York City.

Mr. Potter.

of purification which are too high. Consequently, simple processes for removing the grosser pollution have not been received with favor.

Second.—European experience with the cost of operation and maintenance of fine screens indicates that these items will be exceeded by fully 100% in American practice.

Third.—As most purification works which are projected demand more than one step in the process, fine screens have not seemed capable, for a given cost, of producing results as satisfactory as those obtained with settling tanks of the latest design.

The percentage of suspended matter in the sewage removed by fine screening, as compared with settling, is not a true measure of the relative efficiencies. For the same percentage of suspended matter removed, the screened liquid is far inferior to the settled effluent. The truth of this assertion is amply verified by an examination of Table 4. According to this table the suspended solids before screening amount to 2 112.8, and after screening they amount to 785.2—a reduction of 62.9%; whereas, on the other hand, the residue of the filtered liquid after evaporation has been increased from 1 237.2 to 1 260.4. The speaker is of the opinion that the author's statement, that the organic matter in solution has been increased only 1.9%, is in error. From both quantities should be deducted the total stable solids in solution, these solids being the same in each case. Deducting these, the relation between the organic impurities in solution before and after screening will show a much greater percentage of increase than that stated by the author.

It would seem that in the fine-screening process a portion of the solid matter is converted into colloidal matter in its passage through the screen. As this colloidal matter will pass through filter paper, the screens are credited with a greater percentage of removal than they are entitled to. The transformation of a certain portion of the suspended matter in the sewage into colloids during the process of screening increases the organic impurities in the liquid to a very material extent.

To bear out this statement, the speaker quotes from Mr. John D. Watson's report of 1912 in discussing results obtained at Ashold and Sutton. At this plant, the sewage passes through grit-chambers and receives a preliminary treatment by passing through two Carshalton screens. As to the efficiency of these screens, Mr. Watson says:

"Some time ago one of the Carshalton screens required renewal. This, however, I decided not to do owing to the discovery that the sewage contained more organic impurities *after* it was screened than it did before, thus adding unnecessarily to the work of the bacteria beds. Mr. O'Shaughnessy's report stated that 'Fæcal matter forms a colloidal solution when agitated with water; and the increase in the oxygen-absorbed figure, together with the increase in the colloidal matter present in the sewage liquor after passing the screen, showed con-

clusively the objectionable action of the screen'. The object of eliminating solids is to reduce the strength of the liquid, but to brush particles of fæces over the perforations of the screen and afterwards to force sewage through those perforations must tend to defeat the object in view. The truth of this forced itself upon me in viewing what is perhaps the most complete and elaborate screen I have ever seen, viz., that by Riensch at Dresden." Mr. Potter.

George M. Wisner, M. Am. Soc. C. E., discovered the same phenomena at the Thirty-ninth Street testing station in Chicago, as mentioned in his report of 1911 on "Sewage Disposal for the Sanitary District of Chicago".

It is the speaker's opinion, however, that where dependence for sewage disposal must be placed chiefly on dilution, there is a wide field for fine screening, for in that case the object to be attained is principally the removal of the rather large particles of suspended matter.

Where a high degree of purification is sought, the use of fine screens is of doubtful value. A modern settling tank will give better results and at less cost for a given degree of purification. A settled liquid also is superior to a screened liquid for subsequent biological treatment in filters. Furthermore, a settling tank can handle septic sewage or a sewage in which the suspended matter has become comminuted by pumping or otherwise.

Again, the storing of large quantities of screenings must necessarily be more objectionable than the storing of the digested sludge of a modern settling tank. Not only is the volume of this digested sludge less, but it is also of a less objectionable character.

Settling tanks have a further advantage over fine screens, in the speaker's opinion, in that with them it is possible to omit entirely the use of sludge beds, especially when settling and sterilization provide adequate purification. This can be done by creating a mechanical circulation through the settling tank. The suction pipe for this circulation system should extend down into the digested sludge and the sludge should be raised to a weir box a few inches above the water surface in the settling basin. It should be provided with an adjustable weir to regulate the discharge of digested sludge into the effluent in the proportion that such sludge bears to the volume of sewage flow. The remainder of the digested sludge thus circulated should be returned to the influent pipe, and, as it has a greater specific gravity than the fresh sewage, it will tend to entrain the suspended matter in such sewage, thus resulting in a higher degree of settling in the upper compartment of the tank. The continuous and uniform discharge of digested sludge retains at all times the maximum effective capacity of the sludge digestion chamber and prevents any sudden disturbance in its contents.

Mr. Potter.

The direct discharge of digested sludge, uniformly and continuously, into a body of water with the effluent is unobjectionable from a sanitary standpoint, and will cause less shoaling in the stream than the silt transported in storm-water sewers.

The speaker does not desire to convey the impression that there is no place where fine screens might be given preference. There are many instances, especially when the sewage contains certain trade wastes, whereby the preliminary process of screening—oftentimes of fine screening—is of great value to the subsequent treatment.

In reference to the nuisance created by the composting of large quantities of solid matter removed from fresh sewage, the speaker noted an absence of seriously offensive conditions at the storage yards. At Dresden a great number of chickens had free run over these piles. When the screens were first put into operation, they attracted great numbers of flies during the summer, but after the introduction of the chickens there was a marked absence of flies. No one around the plant had any hesitancy about eating the eggs or the poultry raised on this diet.

Concerning the cost of operation and maintenance of fine screens, the author's data, based on German practice, must be modified, of course, to adapt it to American conditions. The compensation of German labor is less than half that in the United States. As a rule, two shifts are employed in European cities, and three shifts would be necessary in America. Furthermore, it has been the speaker's experience that, as a general rule, the attention given to such plants in Germany cannot be secured in America at any price. It is hoped that this will not always be so. As a result of neglect in attendance, unsatisfactory results are only too often obtained, for which the machinery is blamed.

Screens may be justified because screenings are better fitted as a base for fertilizers than digested sludge. At Dresden, it is proposed to put in a plant for the utilization of the screenings in this way. No less an authority than Mr. John D. Watson has recently reversed himself as to the value of sludge as a base for fertilizers.

In conclusion, the speaker desires to raise this point for discussion: has the action of the German Government in permitting the discharge of settled or screened sewage into the streams of the country been justified? The speaker believes that it has. It has brought about an immediate improvement in all rivers. It has rendered possible the solution of problems otherwise impossible because of the great expense involved in more complete processes.

It is true, of course, that in cities like Birmingham, England, where the sewage flow exceeds the stream flow, more perfect purification is mandatory, but cities thus situated are the exception, especially in America.

Mr. Gregory.

CHARLES E. GREGORY,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Allen's paper supplies the Profession with many necessary data concerning the use of fine screens for the treatment of sewage. In the writer's opinion there is a very extensive field for such screens or other devices which will remove a considerable portion of the suspended solids from sewage which is to be discharged into harbors or rivers having sufficient volume of flow to dilute it materially. The best fine screens undoubtedly remove from fresh sewage a very considerable quantity of materials which are most potent in producing nuisances.

It has been the writer's experience, in most cases where he has had to examine and correct conditions which have given rise to complaints of local nuisance along the water-front of the New York Harbor, that the materials directly causing the nuisance are such as would have been removed by one of the better fine screens. River or harbor waters are capable of digesting without nuisance much greater quantities of sewage from which the coarser and heavier suspended solids have been removed than would be indicated by a comparison of the quantity removed with the total organic matter.

The prevention of the accumulation of material deposits of sewage silt by fine screening or sedimentation has prevented nuisances in practically every case where it has been tried. Large quantities of screened sewage from Dresden and Hamburg are discharged from submerged outlets directly into rivers where the dilution is not great, and apparently the discharge is without detrimental effects. London and Glasgow discharge enormous volumes of effluent, which has been treated by chemical precipitation only, into relatively small streams in which the net outward flow is exceedingly small. All the sewage from London is discharged into the Thames at two points, one on each side of the river, within a relatively short distance from each other. The enormous quantities of organic matter thus discharged reduce the dissolved oxygen content of the river to a very low point in the immediate vicinity of the outfalls, but the river water purifies itself and regains its dissolved oxygen very rapidly, both up stream and down stream from the outfall points. It is reported that the river water is practically as pure 20 miles down stream from the Crossness outfall as it is above London where it is taken for water-supply purposes. Many other instances might be cited to prove conclusively the enormous digestive capacity of waters where the heavier suspended solids have been removed from the sewage.

Recent full-sized experiments in Essen, Germany, have indicated, however, that two-story sedimentation tanks, with retention periods as short as 20 min., will remove from certain sewages about 90% of the matters removable in this way. If it is found that the reten-

* Mt. Kisco, N. Y.

Mr. Gregory.

tion periods for a sewage are short, and if suitable space and location are available, a short-time sedimentation tank will prove considerably more effective and less expensive than a fine screen for removing the suspended matters which have a maximum nuisance-producing effect if allowed to remain. Each particular case, however, will have to be studied and decided on its merits, taking into account:

First.—The nature of the sewage;
Second.—Available space;
Third.—Physical character of the site;
Fourth.—Cost of land;
Fifth.—Relative facility for removing sludge or screenings without nuisance; and
Sixth.—Purity of river or harbor waters required.

Either fine screens or sedimentation tanks may become offensive and objectionable when not properly cared for. The daily removal of screenings is necessary and, if neglected, may cause serious objections in a residential neighborhood.

The lower standards of purity for rivers and harbors, which have been adopted by Germany during the last decade, when compared with corresponding standards established in England, France, and America, during the same period, seem to the writer to have been fully justified by experience, and their practice, therefore, merits adoption by the various authorities of the United States where conditions are similar.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM RICHARDSON BELKNAP, F. Am. Soc. C. E.*

DIED JUNE 2D, 1914.

WILLIAM RICHARDSON BELKNAP, the son of William Burke and Mary (Richardson) Belknap, was born in Louisville, Ky., on March 28th, 1849. He received his early education in the schools of that city, and was prepared for college at the Louisville High School. He then entered the Sheffield Scientific School of Yale University for the select course, and was graduated in the Class of 1869. While at Sheffield, he was a member of Berzelius, captain of the Undine Boat Club, and, in 1869, a member of the *Lit* Board.

After a year's post-graduate course in biology at Yale, Mr. Belknap returned to Louisville, and for three years worked with his father who was in the hardware business there. Subsequently, he spent another year in study in Germany.

On his return to Louisville, he took over his father's business. which at that time was conducted as W. B. Belknap and Company and consisted of one small store dealing almost exclusively in blacksmiths' supplies. Under his management the business grew into the concern known as the Belknap Hardware and Manufacturing Company, employing about 1 000 men, and having representatives in all parts of the United States. Thus, the narrow blacksmith line was broadened out until a description of the Company's manufactures filled a catalogue containing nearly 100 000 items. The present plant consists of modern warehouses with floor space of more than 24 acres and railroad side tracks accommodating 20 freight cars at a time.

In the spring of 1910, Mr. Belknap retired as President of the Company, after having held that office for nearly 30 years. He continued, however, as Chairman of the Board of Directors until his death on June 2d, 1914, following a general breakdown. In his business career he had always kept before himself and his employees the principles advocated by the efficiency engineers of the present day, namely, the elimination of all waste as well as of all waste movements, and he had built up one of the largest businesses in the South from very small beginnings.

Mr. Belknap was a Trustee of Berea College and was active in the support of the Tuskegee and Lincoln Institutes. He was a staunch

* Memoir prepared by the Secretary from information supplied by William B. Belknap, Esq.

supporter of the Kentucky Yale Alumni Association and the Kentucky Scholarship Fund, and was the founder of the William R. Belknap Prizes for excellence in geology and biology in the Sheffield Scientific School.

He was a Director of the Associated Charities of Louisville for nearly 20 years. He was also actively interested in both the Young Men's and Young Women's Christian Associations, and had served as an Elder of the Warren Memorial Presbyterian Church.

Mr. Belknap was for several years a Director of the Louisville Board of Trade, and had served on the Board of Directors of the Southern Exposition. He was one of the founders of the Salmagundi Club of Louisville, a member of the Country Club, and an Honorary Life Member of the Commercial Club, both of that city.

On December 2d, 1874, he was married to Alice Trumbull Silliman, daughter of Professor Benjamin Silliman, Jr., of Yale University. Mrs. Belknap died in November, 1890, and on February 21st, 1894, he was married to Miss Juliet Rathbone Davison, of Louisville, Ky. He is survived by one son and four daughters.

Mr. Belknap was elected a Fellow of the American Society of Civil Engineers on May 28th, 1872.

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THE WATER-PROOFING OF SOLID STEEL-FLOOR RAILROAD BRIDGES.

By Samuel Tobias Wagner, M. Am. Soc. C. E.

To be Presented January 6th, 1915.

Synopsis.

The object of this paper is to submit a specification for the design, materials, and manner of application of the water-proofing of railroad bridges with solid steel floors.

Present practice in bridge construction of this type differs so much among engineers, as to design and materials used, that it is hoped that the discussion from those who have had experience with work of this class will be of value in determining on more uniform practice in a detail of modern bridge construction which is yearly commanding more attention.

The specifications, as presented, have been the outgrowth of about eight years' experience with structures of this class, and with a number of kinds of materials and design. Up to the present time, the specifications which have been used have been tentative, and based on the best information which could be obtained. It seems, however, that there are certain principles—more or less fixed—which may be lost sight of by the inexperienced designer, and the chief object in preparing this paper is to state specifically what they are believed to be, with the hope that the discussion will either refute or approve them.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The specifications are not intended, in any way, to cover the design of the structural work, but to call attention to those details which should be borne in mind by the designer when engaged on a structure, of the type under consideration, which has to be water-proofed. It is believed to be of great importance that the structural designer shall have in mind the fact that the water-proofing plays an important part in the details, and that it should be co-ordinated with the design as he proceeds.

Six Appendices are attached to this paper, for the purpose of adding such information as has been obtained in connection with the details of the subject. Some of the data they contain have not been published heretofore; they have been obtained in an effort to arrive at the truth on lines in which there seemed to be little precedent. Appendix F shows the practice which has been followed in the past six or seven years on work with which the writer has been connected.

The matter thus shown is, briefly, as follows:

Appendix A—Tests of Asphalts;
" B—Examination of Felts and Fabrics;
" C—Examination of Rock Asphalts;
" D—Data as to Use of Asphalts in the United States in 1912;
" E—Abstracts from Paper, by Mr. J. W. Howard, on Asphalts;
" F—Various Plans Used in Actual Practice.

Some explanation is first due as to the reasons for introducing certain features. The water-proofing of solid-floor railroad bridges is undertaken for two reasons:

First.—To protect the metal of the floor from corrosion due to the alternately wet and dry condition of a ballasted floor;

Second.—To prevent the water, which is absorbed by the ballast and given off slowly, from dripping on a street beneath.

A reasonable depth of girders and floor makes a stiff structure, and aids in making it water-tight.

There can be but little doubt that, in all designs, the primary object should be to provide such details as will remove the water from the bridge in the quickest possible time and by the shortest route; and

further, in case of the possibility of the water being held on the bridge for any considerable time after a storm, to design the height of the water-proofing so that it will be above the highest water level. There are many places where the ballast will become very dirty in time, and at such localities this detail is important.

It is also considered inadvisable to throw all the water off the bridge at its ends, over the back walls. It is very difficult to design at this point a satisfactory detail which will prevent water from forcing its way back over the back wall and down the face of the abutment, even though the drainage back of the abutment is specially good. It is much better to collect the water on the deck of the bridge, take it below the floor by inlets and down-spouts, and discharge it below the floor on the ground, or into a sewer (if in a city). Such inlets can be made so as to be accessible from below for cleaning, and, with ordinary care, can be made satisfactory as far as leakage around them is concerned. To carry the water away quickly, there should be a number of these inlets on any bridge of considerable extent.

In the case of deck structures, the water-proofing details are simple, the principal questions to be settled being the type of water-proofing to be used and the durability of the materials. The application of the water-proofing—whatever kind may be selected—is not difficult.

In a half-through structure the conditions for water-proofing are at their worst. If the top of rail is near the top of girder, it is generally advisable to carry the water-proofing over the girder and encase its entire top in concrete. The writer wishes that a number of such girders, in which he has assisted in the design and construction, had been treated in this manner.

The finish of the water-proofing against the web of a half-through girder, the top of which is 3 ft. or more above the top of rail, is one of the most difficult problems to be solved. In some cases, it is the practice to carry the water-proofing nearly if not all the way up to the top of the girder, and protect it with a considerable thickness of concrete, but, generally, it is finished up against the web not far above the level of the top of rail. This finish, if done satisfactorily, will make a good piece of work of the structure as a whole, provided the detail in respect to water running over the back walls is guarded against. It is believed that riveting a flashing angle to the web does not provide a satisfactory detail, as it is impossible to make it continuous on account

of stiffeners and gussets. Placing a pocket of ductile, adhesive mastic (which will not harden in cold weather) as a joint between the water-proofing layer and the web is believed to give the best results, and in case of the deterioration of this asphalt, the detail is in a place where repairs can be made without interfering with traffic. The question of interference with traffic in making water-proofing repairs is one that is giving many railroad bridge engineers serious concern, and is leading slowly, but surely, to much more elaborate, effective, and expensive systems, in order that repairs may be postponed as long as possible. It is possible to make the proper kind of pure asphalt, unmixed with foreign matter, adhere to a clean surface of metal or concrete, and thus secure a tight joint. It has been found advisable to protect this pocket of asphalt with a layer of concrete to prevent the ballast, which may be carelessly thrown against the girder, from cutting into it. (See Figs. 8 and 9, Appendix F.)

As to materials to be used for the solid bridge floor, it is believed that, after the question of durability is disposed of—as one of the most important items—the most durable material, which can be shown to possess adhesion and ductility at low temperatures, and at the same time be sufficiently hard to prevent running at maximum exposure temperatures, is the most desirable—say, temperatures generally between 0° and 200° Fahr. Higher melting points are undesirable, for a number of reasons, and should not be used. The most critical time to test a bridge floor is during melting snow, and cracks which may develop at that time are sure to be channels for water.

Whether felt, fabric, or asphalt mastic shall be used, is a debatable question, and will depend somewhat on conditions. If hard pressed for head room in depth of floor construction, the use of asphalt mastic is indicated, and, though it is generally considered an undesirable material, it is believed that, with proper natural rock, proper flux, and careful mixing and placing, first-class results can be obtained, if the pure asphalt seal is used against the metal. A large bridge, of about 43 000 sq. ft., lately built, is water-tight, and recently the writer has seen materials of this character in splendid condition after 20 years' service.

Felts or fabrics, of course, contain organic matter, and, if not entirely covered with asphalt, are subject to disintegration by "rotting". It would seem, however, that if properly prepared and laid, these

materials should form a very satisfactory membrane—more satisfactory, as far as elasticity is concerned, than the mastic. A felt of inorganic material—asbestos—has all the proper qualities, from the standpoint of durability. It is generally laid in combination with a layer of treated burlap to give it additional strength. Practice sometimes protects asbestos felt with asphalt mastic instead of brick or reinforced concrete. This is probably allowable with asbestos felt, but should never be used with any of the other felts. Experience shows that hot mastic, placed on a layer of felt, or fabric treated with asphalt, will draw the asphalt out of the felt and incorporate it as part of the mastic.

The writer wishes to acknowledge the kind permission of Theodore Voorhees, M. Am. Soc. C. E., Vice-President, and W. Hunter, M. Am. Soc. C. E., Chief Engineer, Philadelphia and Reading Railway Company, and George S. Webster, M. Am. Soc. C. E., Chief Engineer, Bureau of Surveys, City of Philadelphia, to present the data in this paper, and to Mr. Edwin Chamberlain, Assistant Engineer, Philadelphia and Reading Railway Company, and R. J. Lawrence, Assoc. M. Am. Soc. C. E., for assistance in its preparation.

GENERAL SPECIFICATIONS FOR WATER-PROOFING SOLID-FLOOR RAILROAD BRIDGES.

PART I. DETAILS OF CONSTRUCTION.

1.—Depth.—The depth of steel or concrete construction shall be such as to allow a sufficient distance from top of rail to top of steel or concrete floor for proper water-proofing, and protection from the cutting action of the ballast. Under ordinary conditions, a depth of from 3.5 to 4.0 ft. from top of rail to clearance line below is sufficient.

2.—Drainage Grade.—Provision shall be made for grades of at least 1% on the floor of the bridge to remove water promptly. Where this cannot be done in the steelwork, cement concrete, with a minimum thickness of 2½ in., shall be placed so as to drain the water to the inlets.

3.—Inlets.—Cast-iron inlets shall be set at proper places in the floor and provided with movable top grates. The down-spout from each inlet shall be provided with a trap and clean-out, which shall be accessible from below the bridge. The down-spout shall be of wrought iron, and connected to a sewer or arranged according to local conditions.

4.—Details of Steelwork.—Where two longitudinal girders meet over a column, the end stiffeners shall be placed with the outstanding leg toward the center of the girder, and the girders shall be connected with ⅜-in. plates on each side of the web. A plate shall also be placed covering the joint on the top flange.

5.—Stiffening angles on the outstanding edges of gusset-plates in half-through or through girders shall be terminated below the level of the water-proofing, the gusset-plates shall be heavier, and shall be placed from 10 to 12 ft. apart.

6.—Where the top of girder approximates the same height as top of rail, the water-proofing and protection shall cover the entire top of the girder.

7.—The apron-plate from the steel floor over the back wall shall be provided with a curb angle against which to finish the water-proofing, and to this angle shall be riveted a vertical plate to prevent dirt from collecting under the apron-plate. The apron-plate shall be anchored to the back wall.

8.—Water-proofing.—General Design.—On top of the prepared surface of the concrete shall be placed either of the following:

1st.—One or more thicknesses of felt or fabric, of quality and applied as specified hereafter, together with proper protection;

2d.—Asphalt mastic at least 1½ in. in thickness, of quality and applied as specified hereafter.

9.—Felt, Burlap, or Fabric.—When water-proofing of this kind is to be used, either of the following types shall be adopted:

1st.—From four to six layers of felt;
2d.—One middle layer of treated burlap, with four layers of felt;
3d.—One layer of felt, two layers of burlap, and two layers of felt;
4th.—One middle layer of treated burlap, and two layers of asbestos felt;
5th.—Either one or two layers of treated cotton-drill fabric.

10.—Protection of Water-proofing.—After the completion of the felt or fabric water-proofing, the entire surface shall be covered and protected by one of the following methods:

1st.—Straight, hard-burned brick laid flat, with joints filled either with water-proofing compound or cement grout. Water-proofing compound should only be used as a filler on flat or nearly flat surfaces;
2d.—A layer of cement concrete from 2 to 2½ in. thick, with wire reinforcement;
3d.—A layer of about 1½ in. of asphalt mastic used only on top of asbestos felt. It is believed that no protection is necessary for the asphalt mastic.

11.—Special Drainage over Protection.—On top of the protection coat, and outside the line of the ties, a line of half-round cast-iron pipe, 6 in. in diameter, and perforated frequently, shall be placed to collect the water and convey it to the inlets.

Part II. Preparation for Water-Proofing.

12.—Preparation of the Steel.—All openings in the steelwork shall be thoroughly closed, either by caulking with burlap dipped in hot asphalt, or by the use of sheet metal sufficient to maintain the concrete base before applying the burlap and asphalt.

13.—Preparation for Water-proofing Materials.—Wherever called for by the plans, the decks of the bridges shall be protected with 1:3:5 concrete, with ¾-in. stone or gravel, mixed as specified hereafter, finished with a 1:2 mix of cement mortar, ½ in. thick, troweled to a smooth surface on top, as shown. This concrete shall be allowed to dry thoroughly so as to prevent the formation of steam when the hot water-proofing materials are applied.

14.—All vertical or sloping surfaces of concrete or steel shall be thoroughly cleaned of dust, dirt, loose particles, paint, and grease. The use of a hand-bellows is recommended for cleaning loose dust and dirt from the surfaces. For cleaning paint and grease from the steel and freshening the surfaces of asphalt, where a junction of old and new is to be made, or where a pocket of pure asphalt is used against

the girders and the felt or mastic, gasoline shall be used, either by swabbing the surface with it, or by pouring a small quantity over the surface to be cleaned and setting fire to it. The use of a blow-lamp is also recommended.

15.—These surfaces shall then be painted with two coats of approved asphalt, diluted with gasoline. The materials of the first coat shall be proportioned so as to give a brownish tint. The second coat shall have a larger quantity of asphalt.

16.—Both coats of paint shall be thoroughly applied and worked into the surfaces, so as to give a uniform coating of the asphalt.

17.—Paint shall not be applied to damp concrete or steel. The painting shall be done immediately in advance of the application of the water-proofing materials and before dust has had time to collect.

18.—If the concrete is damp before the water-proofing is applied, the surface shall be first covered with a 2-in. layer of hot sand and allowed to stand for from 1 to 2 hours, after which the sand shall be swept back, uncovering sufficient surface to begin work, and the operation repeated over a new surface.

19.—Concrete Proportions.—The cement concrete shall be proportioned by measurement of volumes. The volume of a barrel of cement, 376 lb., shall be assumed to be 3.6 cu. ft. The sand and stone shall not be packed more closely than by throwing, in the usual way, into a barrel or box at the time of measurement.

20.—Cement.—Portland cement shall be used, of the quality specified by the American Society for Testing Materials.

21.—Stone.—The stone shall be clean, hard, crushed stone, or pebbles, to be approved by the Chief Engineer, and shall be composed of the whole run of the crusher, from ¼ in. to ¾ in. in size, screened of dust and particles less than ¼ in. in greatest dimension.

22.—Sand.—The sand shall be clean and sharp, and shall be composed of grains graded from "fine to coarse," screened to reject all particles of a greater diameter than ¼ in. It shall be free from foreign matter, and subject to the approval of the Chief Engineer.

23.—Care of Sand and Stone.—Sand and stone, when delivered on the work, shall be dumped on platforms and not on the ground.

24.—Hand Mixing.—When mixed by hand, the cement and sand shall be first mixed dry and made into a mortar. The stone shall be spread on a suitable floor to a depth of about 6 in., thoroughly wetted, and the mortar evenly spread over it, care being taken that the stone of each batch is mixed as to size. The whole mass shall then be turned over four times and raked, to secure complete and uniform mixture. If the Contractor desires to use some other method, he shall submit it for approval. Should the mixture be permitted to set before placing or tamping, it shall be removed and not used. Hand-mixed batches shall not be larger than 1 cu. yd. in volume.

25.—Depositing.—All concrete shall be deposited as the Chief Engineer shall direct. It shall be of such consistency that when dumped in place it shall not require much tamping, and shall be laid with a view to be an aid to the water-tightness of the structure, and not merely a support for the water-proofing materials. All showing surfaces shall be troweled to a smooth, hard surface.

26.—In cases where concrete haunching against girders is called for by the plans, forms shall be used, and the concrete shall be of a wet consistency.

Part III. Materials and Application.

Water-Proofing Felt or Fabric and Asphalt.

27.—Materials.—On the prepared surface, apply the specified number of layers of approved saturated and coated felt, with a finished surface, and weighing about 14 lb. per 100 sq. ft.

28.—The bids shall be based on the use of the type of felt specified in Paragraph 27, but additional alternate bids will be considered, based on felts or fabrics other than these, which may be approved by the Chief Engineer. In the event of such alternate bids being made, the bidders shall present with them sufficient data as to the methods of manufacture, quality of materials, and references to places where such felts or fabrics have been used, giving dates of application.

29.—All materials shall be delivered on the work in their original packages, and properly branded.

30.—The acceptance or rejection of an asphalt shall rest with the Chief Engineer, and shall be based on the requirements stated in Paragraphs 31 to 37.

31.—The asphalt used shall consist of fluxed natural asphalt, or asphalt prepared by the careful distillation of asphaltic petroleum.

32.—It shall contain, in its refined state, not less than 98% of bitumen soluble in cold carbon-disulphide. The remaining ingredients shall be such as not to exert an injurious effect on the work.

33.—When 20 grammes are heated for 5 hours at a temperature of 325° Fahr., in a tin box 2½ in. in diameter, it shall not lose more than 2% by weight, nor shall the penetration at 77° Fahr., after such heating, be less than one-half of the original penetration.

34.—The melting point shall be between 150° and 190° Fahr.

35.—A briquette of the solid bitumen, having a cross-section of 1 sq. cm. shall show ductility at 40° Fahr., and at a temperature of 77° Fahr. shall show ductility of not less than 20 cm., the material being elongated at the rate of 5 cm. per min. (Dow moulds).

36.—All tests shall be conducted according to methods approved by the Chief Engineer.

37.—The penetration indicated herein refers to the depth of penetration in hundredths of a centimeter of a No. 2 cambric needle, weighted to 100 grammes, at 77° Fahr., acting for 5 sec.

38.—*Application.*—All flashing and reinforcing around inlets and other places specified shall be carefully executed.

39.—Water-proofing shall not be done in wet weather, or at a temperature below 32° Fahr., without special orders from the Chief Engineer. The felt shall be laid shingle fashion, the first two layers longitudinally and the last three transversely to the center line of the bridge, where five layers are called for, and as specified in detail in other cases, and shall be carried up the haunching and made secure against the girder in a satisfactory manner, or as shown on the plans. The flashing against vertical or inclined surfaces shall be in accordance with the directions of the Chief Engineer, if not indicated. The first layer of felt shall not be cemented to the floor of a steel bridge, except around the drain outlets. On an arch bridge, the first layer shall be cemented to the top of the arch. At no point shall there be less than the specified number of thicknesses.

40.—As the hot asphalt is spread, the felt shall be immediately rolled into it, and rubbed and pressed over its surface, so as to eliminate air bubbles and insure thorough sticking. One mopful of the asphalt shall not be spread over more than 1 sq. yd. of surface at one mopping. Not less than 2.5 to 3 gal. of asphalt shall be used on 100 sq. ft. of a single layer of felt. The top layer shall also be mopped and the work done so that the layers shall be one compact mass.

41.—The finish of the water-proofing against the girders or concrete shall be made with a pocket of pure elastic asphalt of the quality specified in Paragraphs 31 to 37, except that the melting point shall be between 140° and 180° Fahr., the ductility at 40° Fahr. shall be at least 3 cm., and the adhesive qualities shall be satisfactory to the Chief Engineer. The surfaces with which this material comes in contact shall be dry, absolutely free from dust or grease, and, previous to its application, shall be covered with a thin paint made by dissolving the asphalt in gasoline, as specified in detail in Paragraph 15.

42.—Particular care shall be taken to make a tight joint around gussets, stiffeners, and the ends of girders.

43.—Care shall be taken to prevent injury in any way to the water-proofing by the passing of men or wheel-barrows over it, or by throwing any foreign materials on it.

44.—After the water-proofing course has been completed, the horizontal surfaces shall be protected, as shown on the plans, by a course of straight, hard-burned and dense brick, laid flat in a bed of 1 to 3 cement mortar, with full joints. There shall be not less than ½ in. of mortar between the felt and the bricks. The brick shall not increase in weight more than 10% when immersed in water for 7 hours.

45.—The haunching, and about 18 in. in width of the horizontal surface adjacent to the haunching, shall be protected, as shown on the plans, by about 2½ in. of 1:3:5 concrete, reinforced with No. 8 and No. 10 wire cloth, electrically welded, having a 3 by 8-in. mesh.

46.—Every care shall be taken to insure satisfactory and thoroughly water-tight joints between the main layer of water-proofing and the girders; and special attention shall be given to stiffeners, gussets, etc. The water-proofing shall also be carried down over the back walls to below the elevation of the bridge seat, as shown on the plans, or as directed.

47.—Rolls of felt shall be stored on end, and not laid on their sides.

48.—Water-proofing shall be done only by experienced and expert felt water-proofers.

Natural Rock Asphalt Mastic.

49.—Rock Asphalt Mastic.—Wherever called for by the plans, the decks of bridges shall be water-proofed with natural rock asphalt mastic, as specified in Paragraph 50.

50.—The cement concrete, prepared as specified heretofore, shall be water-proofed with asphalt mastic equal in quality for the intended purpose, as to ingredients used and resistance to water, to the following specifications, and be approved as such:

Sicilian rock asphalt mastic	60 parts.
Clean, sharp, graded grit and sand to pass a sieve of 8 meshes per inch	30 parts.
Asphalt as specified in Paragraphs 30 to 37	10 parts.

These proportions shall be varied when required by special conditions on the work.

51.—The mixture shall be made at the site of the work, shall be heated to a temperature of from 250 to 300° Fahr., and shall be stirred until all the ingredients are thoroughly incorporated. It shall then be spread and thoroughly worked, to free it from voids, and shall be ironed to a smooth surface with smoothing irons, if so directed. All mastic shall be applied in two coats, making the total thickness shown on the plans. The two coats shall break joints, and the mastic shall be distributed evenly. Where the thickness of the concrete plus mastic is less than 2¼ in., the full thickness shall be made up of asphalt mastic.

52.—All mastic delivered on the work shall be properly branded. Water-proofing shall not be done in wet weather, or at a temperature below 32° Fahr., without special orders from the Chief Engineer.

53.—Pockets of asphalt shall be placed against all metal, and mastic along girders, around stiffeners, gussets, etc., as specified in detail in Paragraphs 14 to 18, inclusive, and 41.

54.—Great care shall be taken around expansion joints, drain-pipes, and similar places, where a separation may take place.

55.—After the mastic is laid, it shall be mopped with pure melted asphalt, and the surface shall be spread with a layer of clean, coarse sand, to harden the top.

56.—The pockets of asphalt placed against the girders, stiffeners, and gussets shall be protected, as shown on the plans, by about 2½ in. of 1:3:5 concrete, reinforced with No. 8 and No. 10 wire cloth, electrically welded, having a 3 by 8-in. mesh.

General Conditions.

57.—General Conditions.—The furnishing and erection of the steelwork for the bridge to be water-proofed will be executed under a separate contract, and the riveting will be completed, the erection finished, and the steel floor cleaned up, ready for the water-proofing, before the work on this contract is begun. In addition to the foregoing, the Contractor shall make a final cleaning of the steelwork before the work of water-proofing is begun.

APPENDIX A.

TESTS OF ASPHALTS. (SEE TABLE 1.)

The following notes are to be considered in connection with the interpretation of the tests recorded in Table 1.

Bitumen Soluble in CS_2.—This represents the total bitumen soluble in CS_2, and is determined by the method recommended by the American Society for Testing Materials. Generally, it may be said that 99.5% of the petroleum asphalts are soluble in CS_2. Others vary, especially the natural asphalts, some of which may run as low as about 55%, although gilsonite will dissolve to the extent of more than 99 per cent.

Bitumen Soluble in 88° P.E.—Hubbard* says:

"The term asphaltenes is commonly applied to bitumen insoluble in petroleum naptha [or ether] and malthenes to that portion which is soluble. It is of course evident that both of these terms cover a multitude of compounds, but in general it may be said that the asphaltenes tend to give body and consistency as well as adhesive properties to the products in which they are found, so that this determination serves as an indication of the mechanical stability of the material as well as its binding qualities."

The figures given are the percentages of the total bitumen thus soluble in 88° P.E.

Ductility.—This test is made by filling with asphalt a mould which has a cross-section of 1 sq. cm., soaking it in water at the desired temperature for 20 min., and then pulling it apart at the rate of 20 cm. per min. The elongation at the moment of rupture is called the ductility, and is expressed in centimeters.†

The ductility at different temperatures indicates the ability of the material to remain firm and elastic under extremes of weather. The less the range of ductility at several temperatures, the better the material for most purposes.

Penetration.—Penetration tests are made by measuring the penetration of a No. 2 standardized weighted needle at different temperatures, and under definite weights and times.

In such tests, the following weights and times are used:

At	40°	Fahr.		200	grammes	acting for	60 sec.
"	77°	"		100	"	" "	5 "
‡"	77°	"	(after heating)	100	"	" "	5 "
"	110°	"		50	"	" "	5 "

The penetration is expressed in hundredths of a centimeter.

* "Dust Preventives and Road Binders," p. 361.

† The details of the method are given in *Proceedings*, Am. Soc. for Testing Materials, 1909, Vol. IX, p. 594.

‡ This test is made after heating for 5 hours to a temperature of 325° Fahr.

TABLE 1—(*Continued*).

Test No.	Bitumen Soluble in:		Ductility, in Centimeters, at:			Penetration at 77° Fahr.			Loss on heating for 5 hours at 325° Fahr.	Flow point, in degrees Fahr.	Melting point, in degrees Fahr.	Remarks.
	CS_2	88° P.E.	40° Fahr.	77° Fahr.	110° Fahr.	40° Fahr.	70° Fahr.	After 5 hours heating at 325° Fahr.				
7-*a*	99.19	.....	0.0	3.0	14.0	11	20	14	0.48%	.....	203	
7-*b*	99.92	70.68	2.5	5.0	.....	32	45	.....	.....	.....	207	
7-*c*	99.86	71.70	3.0	5.5	.....	33	48	.....	.....	.....	201	
7-*d*	99.67	72.11	3.0	5.7	10.5	39	51	40	0.32%	163	192	
7-*e*	99.74	69.51	2.5	5.0	13.5	28	40	40	0.51%	167	198	
7-*f*	99.72	45.94	6.0	6.0	20.0	55	53	34	0.35%	149	190	
7-*g*	99.31	67.32	2.5	6.5	11.0	17	30	25	0.48%	140	183	
8-*a*	99.78	80.27	0.0	45.0	70.0	28	47	21	0.54%	115	150	
8-*b*	99.71	65.34	2.5	7.5	13.0	20	34	23	0.66%	176	205	
8-*c*	99.81	69.95	2.5	5.5	10.0	46	46	30	0.40%	158	201	
8-*d*	99.22	67.42	4.5	8.5	17.2	21	40	30	0.54%	126	171	
9-*a*	98.85	.....	2.0	3.0	4.0	46	57	42	0.47%	.....	228	
9-*b*	98.40	64.64	0.0	1.5	2.0	13	16	15	0.07%	212	260	Cracked after heating 5 hours.
9-*c*	98.72	75.49	2.0	3.0	4.0	51	69	55	0.35%	149	200	
10-*a*	98.98	92.38	2.5	6.0	15.5	31	37	36	0.22%	.....	160	
11-*a*	98.09	64.72	0.0	2.5	4.0	17	20	15	0.35%	.....	244	
12-*a*	99.56	75.14	0.0	10.0	24.0	28	50	29	0.33%	140	156	
13-*a*	96.74	.....	2.3	3.0	.....	38	42	.....	.....	.....	252	
13-*b*	99.74	52.25	4.0	14.0	30.0	50	64	38	0.44%	133	163	
13-*c*	99.71	67.38	5.0	18.5	48.0	50	64	32	0.63%	122	149	
13-*d*	99.51	74.62	5.5	8.5	22.0	33	60	47	0.09%	129	158	
13-*e*	98.62	80.76	5.00	9.00	38.0	34	69	56	0.20%	117	163	
14-*a*	99.82	67.40	0.5	1.5	3.7	32	27	25	0.43%	212	266	
14-*b*	99.57	66.30	0.5	1.0	1.8	36	26	20	0.47%	250	297	
14-*c*	99.64	75.43	3.5	7.5	11.2	50	64	54	0.27%	167	196	
14-*d*	95.88	75.76	4.00	6.5	11.0	31	57	49	0.25%	122	172	
15-*a*	99.80	70.79	2.5	3.5	5.0	43	53	43	0.44%	203	230	
16-*a*	61.76	59.21	0.0	0.5	8.0	0.5	4	1	1.22%	200	225	Refined Trinidad.
17-*a*	93.00	75.33	0.0	24.5	37.5	3	20	8	2.68%	130	145	Refined Bermudez.
18-*a*	99.90	67.57	0.5	2.3	3.5	26	54	42	0.70%	167	199	

TABLE 1.—TESTS OF WATER-PROOFING ASPHALTS.

Test No.	Bitumen Soluble in:		Ductility, in Centimeters, at:			Penetration at 77° Fahr.			Loss on heating for 5 hours at 325° Fahr.	Flow point, in degrees Fahr.	Melting point, in degrees Fahr.	Remarks.
	CS_2	88° P.E.	40° Fahr.	77° Fahr.	110° Fahr.	40° Fahr.	77° Fahr.	After 5 hours heating at 325° Fahr.				
1-*a*	99.48	.....	2.0	3.0	3.5	40	36	26	0.12%	.....	.....	
1-*b*	99.88	.....	2.5	3.0	.....	18	40	.....	0.38%	.....	266	
1-*c*	99.91	67.40	.....	2.5	.....	28	29	28	0.31%	230	270	
1-*d*	99.83	63.11	1.0	3.5	4.0	29	28	21	0.45%	244	275	
1-*e*	99.73	60.43	0.75	3.5	4.5	15	20	15	0.65%	194	228	
1-*f*	99.92	61.89	0.75	3.75	5.25	16	23	22	0.89%	154	228	
1-*g*	99.74	66.04	1.00	2.00	3.5	33	41	32	1.90%	176	212	
2-*a*	98.33	.....	1.00	too	soft	54	163	70	0.27%	.....	.....	
2-*b*	99.88	54.37	4.6	over	100	18	over 180	180	0.11%	72	91	
3-*a*	97.41	.....	1.5	2.0	3.5	37	33	24	0.72%	.....	.....	
3-*b*	95.89	24.62	0.0	4.2	too soft	35	86	33	2.99%	.....	119	
3-*c*	99.04	36.12	3.0	3.5	1.5	38	46	36	1.11%	194	234	
3-*d*	98.93	36.28	2.5	3.0	1.5	40	43	37	0.72%	216	250	
4-*a*	99.37	.....	3.0	8.0	.....	13	49	35	0.30%	.....	180	
5-*a*	99.41	.....	0.5	2.5	3.0	18	24	14	0.25%	.....	250	
5-*b*	98.97	.....	3.5	8.0	17.0	42	57	40	0.68%	.....	169	
5-*c*	98.95	.....	4.0	13.0	.....	41	49	49	0.14%	.....	168	
5-*d*	99.38	.....	4.0	17.0	.....	19	48	36	0.17%	.....	185	
5-*e*	99.80	.....	.....	9.0	.....	25	60	38	0.23%	.....	176	
5-*f*	99.60	68.97	4.0	7.0	18.0	31	49	32	0.26%	138	155	
5-*g*	99.33	60.09	0.0	2.5	3.5	12	15	15	0.03%	212	262	
5-*h*	99.84	68.72	0.0	8.5	27.5	37	44	32	0.19%	158	187	
5-*i*	99.56	69.94	0.0	15.5	30.0	31	47	35	0.36%	131	169	
5-*j*	99.66	70.42	3.0	9.0	42.0	35	40	30	0.12%	113	163	
5-*k*	99.64	70.77	3.5	17.0	100.0	28	49	34	0.50%	149	176	
5-*l*	99.71	67.93	3.5	12.0	53.0	31	42	30	0.36%	147	174	
5-*m*	100.00	85.45	4.0	12.0	50.0	14	35	23	.....	140	153	
5-*n*	99.66	67.20	2.0	2.5	3.0	40	43	38	1.33%	217	248	Identity of sample questioned.
5-*o*	99.96	65.20	4.0	8.5	16.0	24	28	25	0.67%	131	225	
5-*p*	99.78	74.14	4.5	29.5	68.0	39	53	31	0.63%	117	146	
6-*a*	99.59	69.97	4.5	10.5	26.0	32	42	26	0.37%	149	176	
6-*b*	99.75	56.78	4.0	19.0	51.0	46	51	29	0.10%	131	163	
6-*c*	99.78	71.70	7.0	53.0	100.0	32	52	32	0.72%	131	158	
6-*d*	99.92	85.45	4.5	12.0	50.0	14	35	23	.....	140	153	

Loss on Heating.—This is a volatilization test, and is made by heating a sample in the air oven for 5 hours at a temperature of 325° Fahr., and then expressing the loss as a percentage. It is believed, by investigation of the subject, that the loss of weight thus ascertained is a fair comparative indication of the loss by volatilization suffered in the course of time on exposure. It is made in a similar manner to that described by Richardson.*

Melting Point.—This is determined by preparing ½-in. cubes of the material, and measuring the temperature.†

The melting point of an asphalt is very closely related to its hardness or brittleness, but varies considerably. The determination of the proper melting point depends, almost entirely, on the conditions under which the material is to be used and how it is to be mixed and applied. High melting point materials cannot be used to apply in thin layers to cold materials, on account of the rapidity with which they cool, but can be used in mastic mixtures, if the other ingredients are hot. It is generally true that materials with a high melting point will be deficient in ductility at low temperatures.

In the tables of tests, the numbers 1, 2, etc., represent different brands of prepared asphalt; the letters "a", "b", etc., refer to different samples.

* "The Modern Asphalt Pavement."

† By the method described in *Proceedings*, Am. Soc. for Testing Materials, 1909, Vol. IX, p. 594.

APPENDIX B.

FELTS AND FABRICS. (SEE TABLE 2.)

For use in solid bridge floor water-proofing, there are a large number of felts, burlaps, and fabrics, which are treated in the shops with various asphaltic compounds, and are used to make up a compound membrane in the field by fastening the several layers together with some asphalt compound applied hot.

The tests in Table 2 are representative of such materials, as prepared by nine different manufacturers. They are intended to show the percentages of bitumen and felt or fabric per square foot, as well as the total bitumen, and its solubility in 88° P.E.

TABLE 2.—TESTS OF WATER-PROOFING FELTS AND FABRICS.

No.	Description.	Percentage of Bitumen.		Original weight, in ounces per square foot.	Bitumen, per Square Foot.		Felt, Etc., per Square Foot.		Tensile strength, in pounds per square foot.
		Total.	Percentage soluble in P.E.		Ounces.	Percentage.	Ounces.	Percentage.	
1-*a*	Saturated and coated burlap	71.47	63.78	2.88	2.06	71.47	0.82	28.53	60
1-*b*	Saturated felt	57.76	73.55	2.31	1.33	57.76	0.98	42.24	20
1-*c*	Saturated and coated felt	72.81	55.39	2.32	1.69	72.81	0.61	26.51	25
2-*a*	" " " "	61.13	73.06	2.29	1.40	61.13	0.88	38.39	30
2-*b*	" " " burlap	61.15	52.65	2.19	1.45	66.20	0.74	33.80	58
2-*c*	" " " felt	56.35	47.25	2.66	1.66	62.40	1.00	37.60	18
2-*d*	" " " "	57.90	73.40	2.74	1.59	58.00	1.15	42.00	
3-*a*	" " " "	66.69	69.92	2.24	1.49	66.69	0.74	33.14	35
3-*b*	" " " "	74.05	58.60	2.54	1.98	77.90	0.56	22.10	20
4-*a*	" " " "	58.88	69.53	2.75	1.62	58.88	1.03	35.52	60
4-*b*	" " " burlap	58.80	.....	2.43	1.43	58.75	1.00	41.25	
4-*c*	" " " felt	52.48	.....	1.90	0.99	52.50	0.90	47.50	
5-*a*	Saturated asbestos felt	.....	.....	2.84	1.72	60.50	1.12	39.50	
5-*b*	Saturated burlap	.....	.....	1.04	0.69	66.90	0.35	33.10	
5-*c*	" "	52.78	77.85	1.49	0.79	53.02	0.70	46.98	
5-*d*	Saturated asbestos felt	27.17	76.81	1.86	0.51	27.42	1.35	72.58	
6-*a*	Saturated felt	53.2	78.25	2.26	1.20	53.20	1.06	46.80	44
7-*a*	" "	59.35	.....	2.61	1.55	59.40	1.06	40.60	
8-*a*	" "	59.20	73.83	2.72	1.61	59.20	1.11	40.80	
9-*a*	Saturated fabric	50.77	69.06	1.28	0.63	49.20	0.65	50.80	
9-*b*	" "	54.70	41.15	1.28	0.57	44.50	0.71	55.50	
9-*c*	" "	62.50	63.41	1.38	0.86	62.30	0.52	37.70	
10-*a*	Saturated burlap	61.00	60.50	2.31	1.41	61.04	0.90	38.96	80
10-*b*	" felt.....	58.25	73.80	2.40	1.40	58.33	1.00	41.66	28

The felts are generally composed of wood or vegetable fiber, with a little wool in some cases, and in one case, asbestos fiber.

As to finish, there seem to be generally two kinds of these felts: 1st, the felt simply saturated with the asphalt compound, with what might be called a rough finish; 2d, a saturated and coated felt with the finish smooth and hard. Of these, the first is much the cheaper.

Table 2 shows that though there are considerable differences in the quantity of bitumen per square foot, expressed in the form of a percentage, the actual differences are not very great.

The fabrics alluded to are of two kinds: 1st, burlap; 2d, some kind of closely-woven cotton material. These are saturated with the asphalt compound during manufacture. Burlap should never be used in an untreated state on the work, as the raw material is not easily embedded in the hot asphalt in the field, and, further, it takes up water readily from the air. The temperature of the asphalts used should not be greater than 400°, as higher temperatures are likely to burn the burlap.

The figures, 1, 2, etc., in Table 2, represent different manufactures, and the letters, *a*, *b*, etc., refer to different samples.

APPENDIX C.

Rock Asphalts. (See Table 3.)

The only rock asphalts here treated are the imported materials from Switzerland, Germany, and Sicily. The rock, as imported from these localities, contains in itself the following quantities of bitumen.*

Ragusa, Sicily.	Seyssel. France.	Verwohle.	Sicula. Sicily.	Neuchâtel. Switzerland.	Mons.
9.9%	5.9%	7.5%	10.2%	9.1%	8.9%

This rock is ground and fluxed with additional asphalt, so that, when poured into cakes and shipped to the site of the work, the percentages of bitumen vary from 13 to 18%, and it is this material which is again melted, with the addition of about 10% of flux, and incorporated with grit or small pebbles to make the mastic.

The figures in Table 3 give the results of a number of analyses of two brands of asphalt mastic. In a few cases, the granularimetric analysis of the natural rock freed from the bitumen is given. It will be noted that this runs very high in the finest particles of the limestone, which is the natural rock impregnated in both kinds of asphalts tested.

TABLE 3.—Analysis of Rock Asphalts as Delivered on the Work.

No.	Total bitumen soluble in CS_2.	Percentage of total bitumen soluble in 88° P. E.	Percentage of insoluble organic matter.	Percentage of moisture.	Percentage of ash.	Granularimetric Analysis. Passing sieves: No. 200.	No. 80.	No. 40.	No. 10.	Retained on: No. 10.	Remarks.
1-*a*	16.70	63.22	.....	.....	.....	46.3	28.9	8.0	0.1	0.0	
1-*b*	17.40	68.96	.....	.....	.....	51.4	24.6	6.1	0.5	.....	
1-*c*	16.90	61.90	.....	.....	.....	52.6	19.2	9.9	1.4	0.0	
1-*d*	14.47	.....	10.95	0.12	74.46	.....	.....	.....	.....	.....	
1-*e*	16.52	65.76	30.66	0.02	52.80	.....	.....	.....	.....	.....	
1-*f*	17.93	62.80	3.01	0.21	78.85	.....	.....	.....	.....	.....	
1-*g*	18.07	68.42	12.46	0.20	69.27	.....	.....	.....	.....	.....	
1-*h*	11.06	67.03	39.32	0.02	49.60	.....	.....	.....	.....	.....	
2-*a*	16.76	60.12	2.03	0.09	81.12	.....	.....	.....	.....	.....	
2-*b*	17.70	59.48	.....	.....	.....	59.8	13.0	9.2	0.3	0.0	
2-*c*	18.07	67.29	5.11	0.00	76.82	.....	.....	.....	.....	.....	
2-*d*	15.90	.....	6.7	.....	.....	.....	.....	.....	.....	.....	
2-*e*	16.17	.....	12.79	0.42	70.62	.....	.....	.....	.....	.....	
2-*f*	13.42	93.82	12.21	0.09	74.28	.....	.....	.....	.....	.....	
2-*g*	15.9	.....	.....	.....	70.44	.....	.....	.....	.....	.....	74.1% calc. rock powder.
2-*h*	13.27	.....	16.14	0.15	.....	.....	.....	.....	.....	.....	
3-*a*	17.8	67.04	.....	.....	.....	56.3	20.6	5.1	0.2	0.0	

Records, in a number of cases, of long life for these imported rock mastics, make them very desirable for water-proofing purposes under certain conditions. Cases of more than 20 years' use, with the material apparently in as good condition as it was when placed, are known.

* "The Modern Asphalt Pavement," by Clifford Richardson, M. Am. Soc. C. E., p. 253.

APPENDIX D.

MINERAL RESOURCES OF THE UNITED STATES FOR 1912.
PRODUCTION OF ASPHALT BY VARIETIES.

	Short tons.	Percentages.
Bitumen rock	53 041	11.7
Refined bitumen	22 852	5.1
Maltha	474	0.1
Wurtzilite (elaterite)	8 452	1.9
Gilsonite	31 478	7.0
Oil asphalt	333 213	74.2
Total	449 510	100.0

The total imports of asphalt in 1912 were 218 328 short tons.

APPENDIX E.

Extracts, from an Article Entitled "Asphalt Paving Cements and Road Binders", by Mr. J. W. Howard.*

Mr. Howard's article, though written with reference to roads and pavements, has much of interest in connection with the subject of the use of asphalts for water-proofing purposes, and the writer calls special attention to it in order that the various features may be elaborated by those interested in the subject, and ultimately form the basis of more specific data for water-proofing specifications.

What follows has been abstracted, briefly, with the object of showing some of the properties which are considered necessary in an asphalt, and without giving the laboratory methods, which are described in each case.

1. Adhesiveness, or Cementing Strength.—This is a self-evident property needed to cement together the particles of a mastic or the various layers of felt.

2. Water-proofness, or Freedom from Injury by Water.—This is important on account of the exposure of the materials to water moisture, either directly or through capillary action. The formulation of some test which can be made readily to determine this property is of special value, as it is well known that asphalts differ in the property of resisting the action of water.

3. Immutability, or Freedom from Deterioration on Exposure to Sun and Air.—This is important on account of the bad effects which may result from the heat of the sun, the action of the air, and other elements. The asphalt must retain its life and qualities. This test is made by heating, as called for in the specification presented, and noting the percentage of loss; also in examining the penetration before and after the heating.

4. Ductility.—Ductility is the ability to yield without breaking, and is an important property in asphalt used in solid-floor bridges. It is also necessary that the material should be able to contract from cold without cracking. The data for this property are embraced in the specifications, and have been studied more or less carefully. A reasonable and consistent ductility at various temperatures is a desired property.

5. Flexibility or Pliability.—Flexibility or pliability without brittleness is needed for practically the same reasons as are given for ductility, and because the asphalt should be able to yield without cracking. Very little data are available concerning this property.

6. Cohesiveness.—This is the property needed by an asphalt in order to prevent it from pulling apart or cracking within itself,

* *Engineering Record*, September 27th, 1913.

although it may adhere firmly to the materials which it is to bind together. This property is one that has not been studied in water-proofing materials, but is deserving of further care and study.

7. Malleability.—This quality is referred to by Mr. Howard, but is not required for water-proofing materials.

8. Consistency.—This is the right kind of hardness or softness necessary under various temperature changes. It is also known technically as penetration. The tests of the various asphalts and the methods of making the tests are given in Appendix A.

9. Minimum Susceptibility to Softening or Stiffening During Extreme Weather Temperatures.—All asphalts are softened by hot and stiffened by cold weather. Thus far it has been impossible to devise asphalts which will not do this. All good ones, however, do not become too soft or fluid in hot weather, or too stiff in cold weather. This is shown by penetrations at freezing and at high temperatures. (See Appendix A.)

10. Purity or High Percentage of Bitumen.—This quality is important, for several reasons. It excludes foreign matter, which is often injurious. Useless matter is not cementitious, and reduces the quantity of work that can be done with a given quantity of pure asphalt.

11. Freedom from Injury by Heat.—Because all asphalts must be heated sufficiently to apply in the work, it is important that they should not be injured by being heated and maintained at a fluid temperature for a reasonable time. The same test as for immutability may be used.

APPENDIX F.

Plans of Different Methods of Water-Proofing.

Figs. 1 to 9 illustrate progress in water-proofing design, and should be considered and studied in the order in which they are numbered, as the sequence represents what are considered to be improvements in each case.

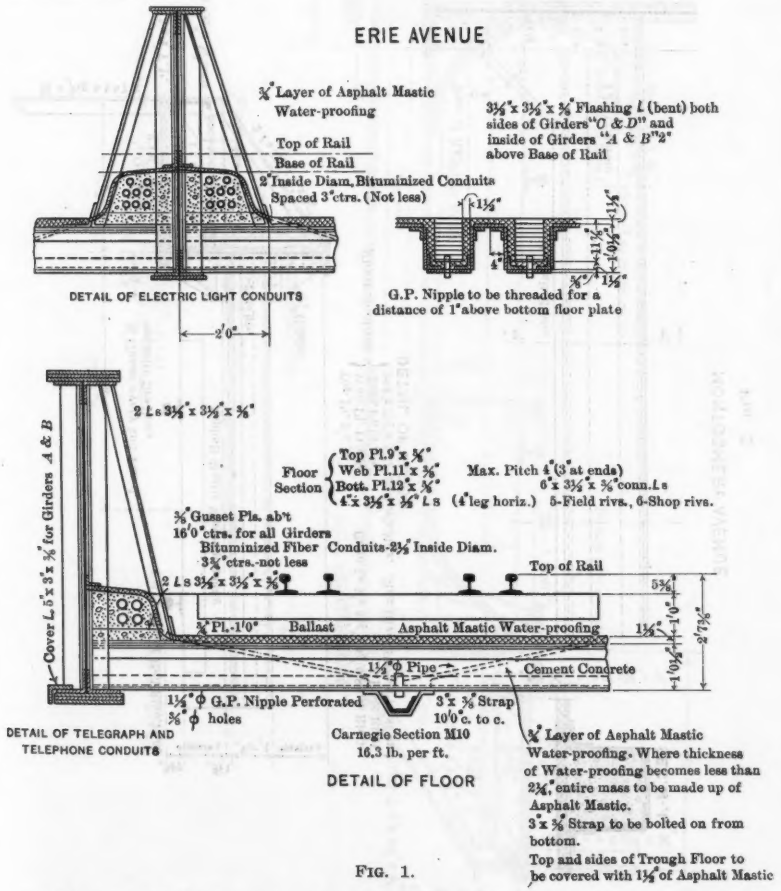
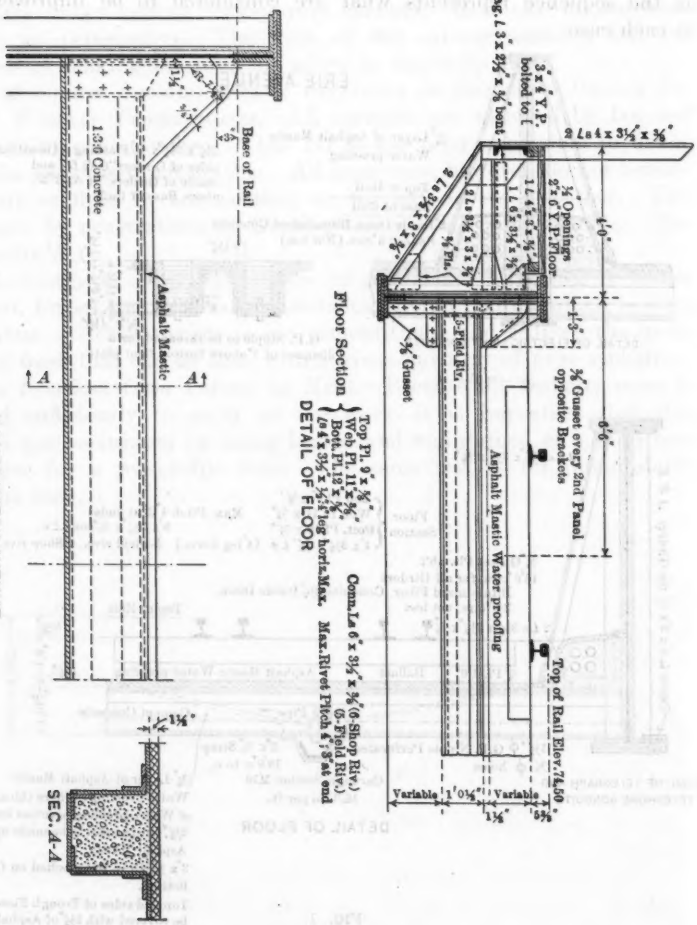


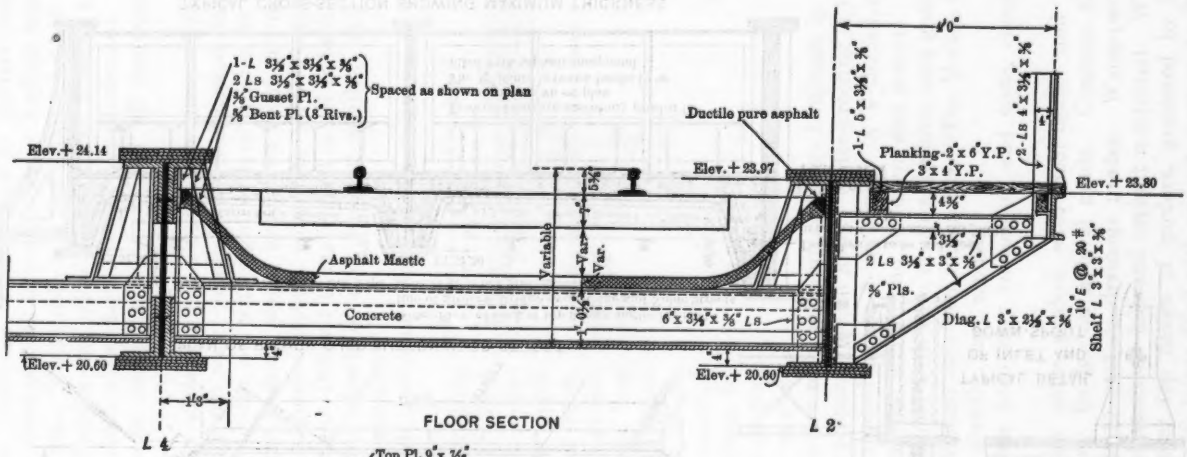
FIG. 1.

Fig. 1.—Half-through plate girder, with trough floor, with flash angles along webs between stiffeners and gussets. Water-proofing of asphalt mastic placed around entire surface of troughs. Each trough provided with nipple in center, to which floor slopes. Concrete used



MONTGOMERY AVENUE

Fig. 2.



FLOOR SECTION

For Panels 12'1½" c.c. Girs.
- Top Pl. 9"x 7/16"
- Web 11"x ⅜" (111/4# b.b. Ls)
- Ls 4"x3½"x⅜" (Riv. Pitch-3" at ends and 4" at center)
- Bott. Pl. 12"x 7/16"

For Panels 14'0¾" c.c. Girs.
- Top Pl. 9"x ⅜"
- Web 11"x ⅜" (111/4# b.b. Ls)
- Ls 4"x3½"x½" (4" leg horizontal)
- Bott. Pl. 12"x ⅜"

ARAMINGO AVENUE

FIG. 3.

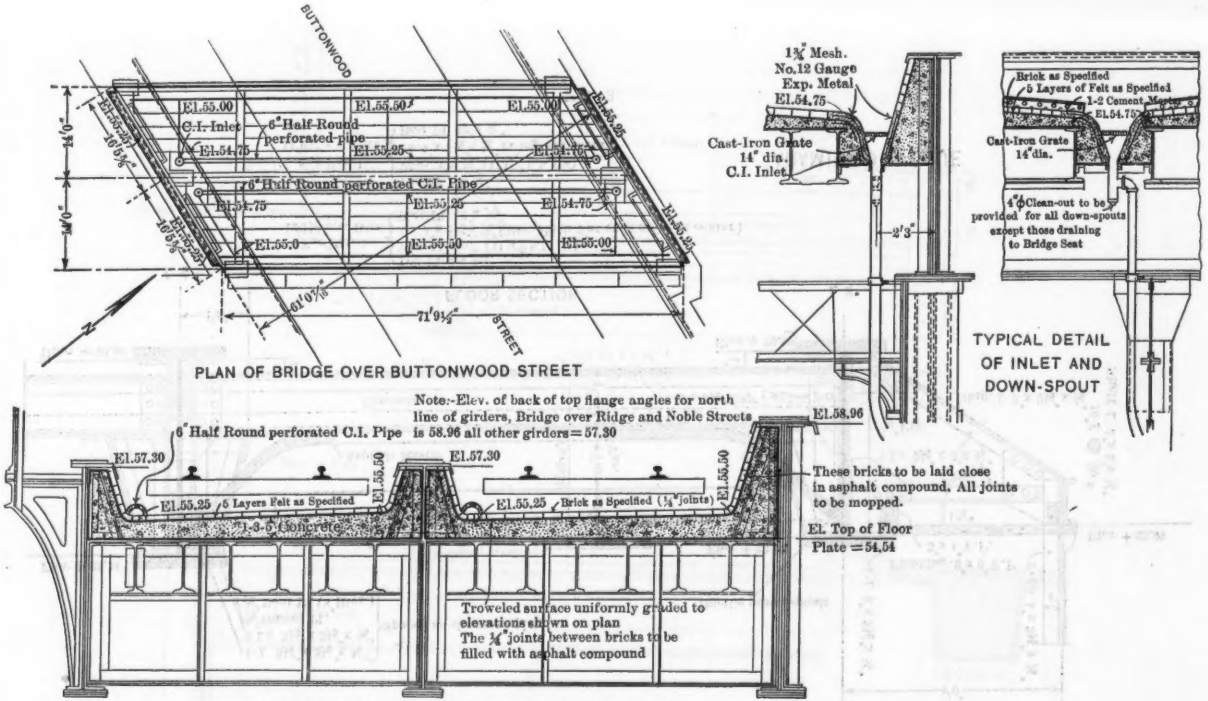

BUTTONWOOD
El.55.00
C.I. Inlet
El.54.75
6" Half-Round perforated pipe
El.55.25
El.55.50
El.55.00
6" Half Round perforated C.I. Pipe
El.54.75
El.55.25
El.55.0
El.55.50
El.55.00
El.55.25
24'-0"
16'-0"
16'5⅝"
61'0⅞"
71'9½"
STREET
N
PLAN OF BRIDGE OVER BUTTONWOOD STREET
1¼" Mesh. No.12 Gauge Exp. Metal
El.54.75
Cast-Iron Grate 14" dia.
C.I. Inlet
2'3"
Brick as Specified
5 Layers of Felt as Specified
1-2 Cement Mortar
El.54.75
Cast-Iron Grate 14" dia.
4" Clean-out to be provided for all down-spouts except those draining to Bridge Seat
TYPICAL DETAIL OF INLET AND DOWN-SPOUT
Note:-Elev. of back of top flange angles for north line of girders, Bridge over Ridge and Noble Streets is 58.96 all other girders = 57.30
El.58.96
6" Half Round perforated C.I. Pipe
El.57.30
El.57.30
El.55.50
El.55.50
El.55.25
5 Layers Felt as Specified
1-3-5 Concrete
El.55.25
Brick as Specified (¼" joints)
These bricks to be laid close in asphalt compound. All joints to be mopped.
El. Top of Floor Plate = 54.54
Troweled surface uniformly graded to elevations shown on plan
The ¼" joints between bricks to be filled with asphalt compound
TYPICAL CROSS-SECTION SHOWING MAXIMUM THICKNESS OF WATER-PROOFING

FIG. 4.

as filler beneath mastic. Drainage beneath floor by gutters leading to down-spouts and into street gutters beneath bridge. Asphalt beneath flash angle and against web. The nipple is a bad detail. Asphalt will break loose from web of girder and allow water to seep through along web at stiffeners and gussets when flash angle is not continuous.

The cost of such work, including concrete and all drainage, is about $0.25 per sq. ft.

Fig. 2.—Half-through plate girder, with trough floor. Troughs filled with 1:3:6 concrete and water-proofed with rock asphalt mastic. Drainage to one end of bridge, or, with light or level grades, to both ends, leading to dry stone packing placed against backs of abutments. No special precaution to make asphalt adhere to girder web—will crack and let water through. Difficult to make good detail at back wall, causing water to back up, especially after freezing weather, and run on bridge seat and down abutment.

The cost of such work, including concrete and drainage, is about $0.35 per sq. ft.

Fig. 3.—Half-through plate girder, with trough floor. Very shallow girder shown in drawing, but same principle applicable to girders of greater depth. In case of proportions shown, probably it would have been wiser to have encased and water-proofed entire top flange of girder. Before applying concrete to fill troughs, all openings between girders and ends of floor sections are carefully caulked with burlap dipped in a low-melting-point asphalt. Drainage is over back wall with specially prepared, sealed joint. Water-proofing is 1½ in. of rock asphalt mastic applied in two ¾-in. layers. A V-shaped opening left along the girder web, and around stiffeners and gussets. The sides of this opening cleaned with wire brushes, bellows, and gasoline, and then painted with asphalt dissolved in gasoline. The V then filled with low-melting-point, ductile, pure asphalt.

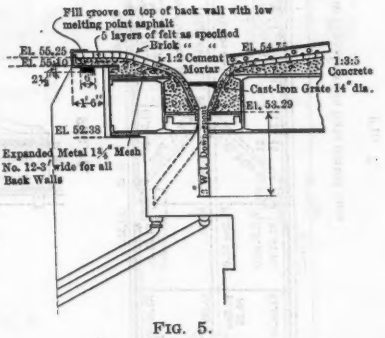


FIG. 5.

This type of floor has been very successful, and costs, complete, about $0.45 per sq. ft.

Fig. 4.—Half-through plate girders with floor-beams and with I-beam and plate floor. Bridge on a level grade. Concrete filling of 1:3:5 mix, sufficient to produce drainage grades. Water-proofing consists of five layers of surfaced felt and asphalt compound. Water-proofing carried up under top flange of girders, protected by layer of hard brick, laid flat, with joints filled with asphalt compound. Water

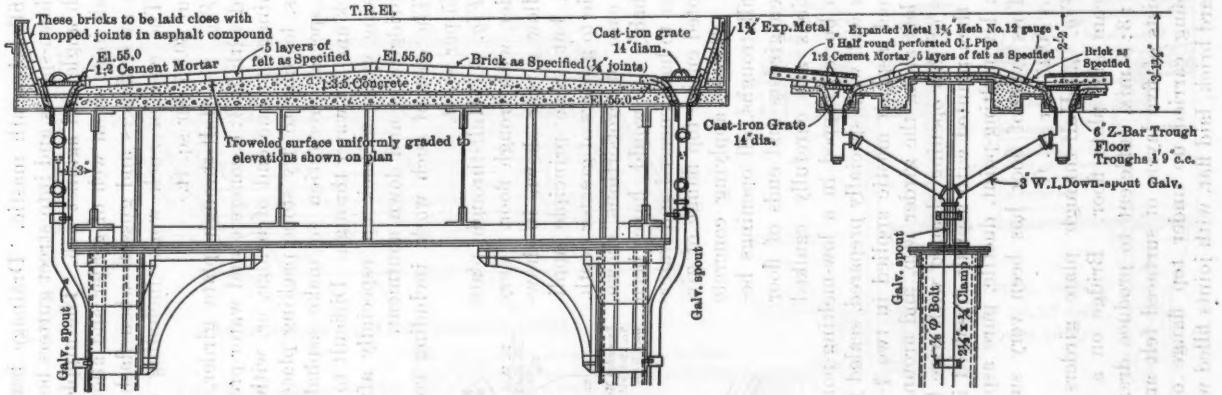

These bricks to be laid close with mopped joints in asphalt compound
T.R.El.
El.55.0
1:2 Cement Mortar
5 layers of felt as Specified
El.55.50
Brick as Specified (¼" joints)
Cast-iron grate 14" diam.
1:3:5 Concrete
Troweled surface uniformly graded to elevations shown on plan
3"
Galv. spout
Galv. spout
1¼" Exp. Metal
Expanded Metal 1½" Mesh No.12 gauge
6" Half round perforated C.I. Pipe
1:2 Cement Mortar
5 layers of felt as Specified
Brick as Specified
3'-10½"
Cast-iron Grate 14" dia.
6" Z-Bar Trough Floor Troughs 1'9" c.c.
3" W.I. Down-spout Galv.
Galv. spout
Bolt
Clamp

FIG. 6.

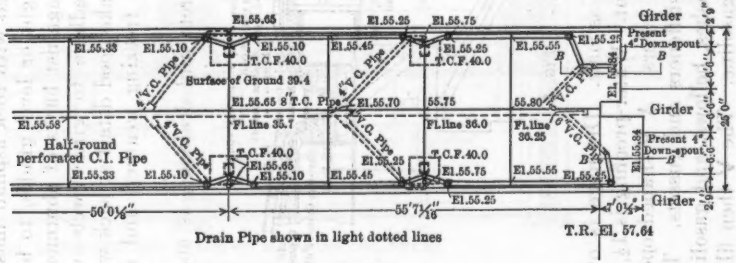

El.55.65
El.55.25
El.55.75
Girder
Present 4" Down-spout
El.55.33
El.55.10
El.55.10
T.C.F.40.0
Surface of Ground 39.4
El.55.65
8" T.C. Pipe
El.55.70
55.75
55.80
Girder
El.55.58
Fl.line 35.7
Fl.line 36.0
Fl.line 36.25
Half-round perforated C.I. Pipe
T.C.F.40.0
El.55.65
El.55.10
El.55.45
El.55.25
El.55.75
El.55.55
Girder
50'0½"
55'7 3/16"
El.55.25
1'0½"
Drain Pipe shown in light dotted lines
T.R. El. 57.64

FIG. 7.

drains to inlets placed near ends of bridge and graded so as to prevent any water from flowing over the back walls. Drainage through grates and clean-outs into down-spouts to sewer. Half-round, perforated, cast-iron pipe placed on top of brick to assist flow of water if ballast becomes dirty. Another detail of back-wall drainage, shown in Fig. 5, has been very successful.

Costs about $0.75 per sq. ft. High on account of grade of track being level, requiring additional concrete and inlets to provide proper drainage.

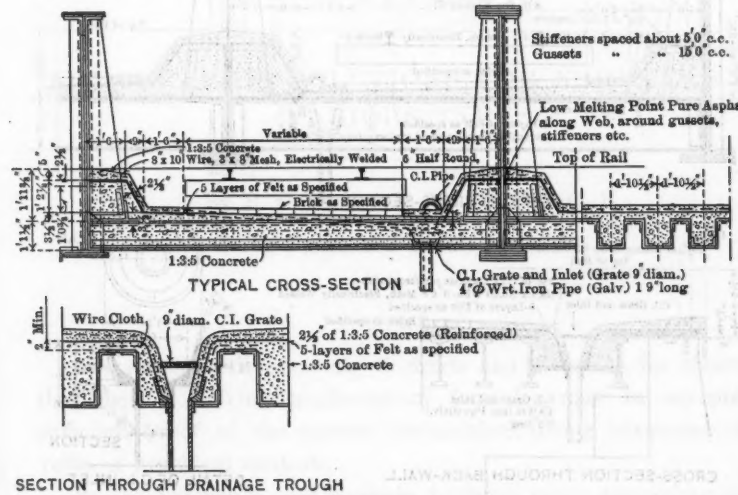


FIG. 8.

Figs. 6 and 7.—Deck viaduct with shallow-trough floor. Tracks level. Troughs filled with 1:3:5 concrete and water-proofing of same character as in Fig. 4. Drainage shown in Fig. 7. Only improvement suggested is use of cement mortar in place of asphalt for brick protection. Very successful.

Cost, $0.65 per sq. ft. High on account of level grade on viaduct, requiring additional concrete and inlets to provide proper drainage.

Figs. 8 and 9.—Half-through plate girders. Fig. 8 with trough floor and Fig. 9 with I-beams and plates. Filling with 1:3:5 concrete. Water-proofing, five layers of surfaced asphalt felt with asphalt compound, and protected with hard-burned brick in 1:3 cement mortar on flat floor and 1:3:5 concrete, reinforced with No. 8 and No. 10 wire, 3 by 8-in. mesh, electrically welded in gutters and over haunching. Pocket of low-melting-point, pure asphalt, carefully put in as

seal against girders and around stiffeners and gussets before protection is placed. Drainage to inlets. No water going over back walls. Water removed from copings by special inlets built in masonry.

Under contract and not completed at present time. Cost about $0.62 to $0.80 per sq. ft., including all drainage details.

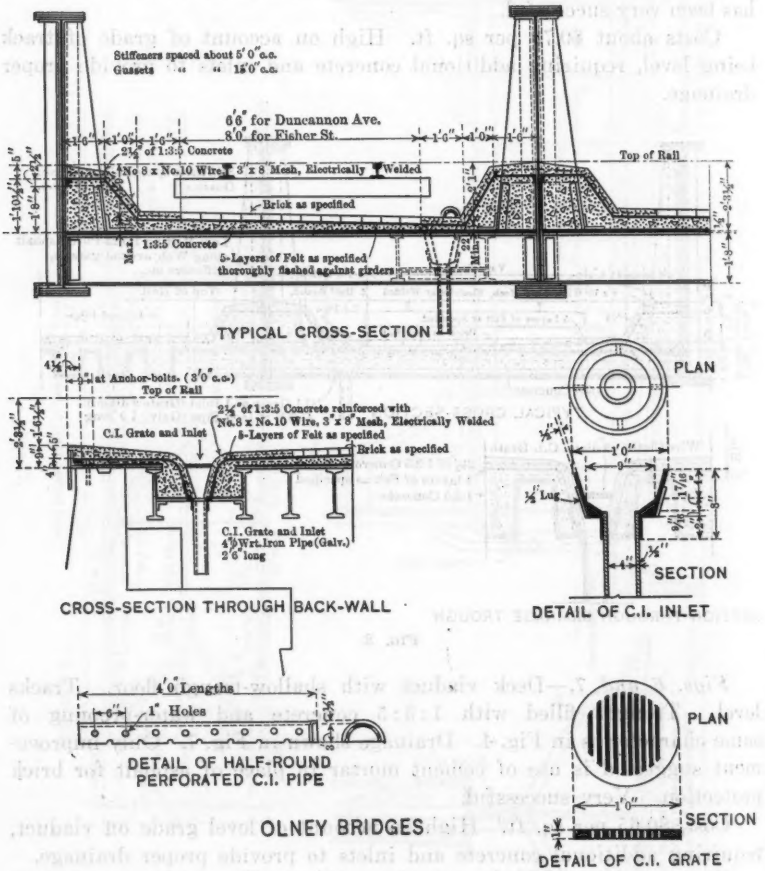


FIG. 9.

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PAPERS AND DISCUSSIONS

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NOMOGRAPHIC SOLUTIONS FOR FORMULAS OF VARIOUS TYPES.

By R. C. Strachan, M. Am. Soc. C. E.

To be Presented February 3d, 1915.

Synopsis.

The present extensive use of charts and diagrams for shortening the labor of solving mathematical formulas may be accepted as sufficient proof of the general recognition among engineers of the value of graphical methods.

By the commonly used methods, however, many expressions cannot be represented without the use of a large number of intersecting lines, which cause the chart to be more or less confusing and difficult to read.

This paper shows the application of the nomographic principle of representation to a number of extensively used formulas, presenting illustrative charts and detailed descriptions of their construction.

The underlying principle is that of plotting the scales representing the variables in such a way that a line crossing the scales anywhere gives a set of readings which satisfy the equation. This principle has not received from American engineers that degree of attention to which it is entitled by reason of its great value and wide range

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

of applicability, and the object in presenting the paper is to call attention to this somewhat neglected labor-saving device.

A nomographic representation, or, more concisely, a nomograph of a formula, may be defined as a plot or chart on which appear scales for the variables involved in the formula, their relative magnitudes and relative positions being such that corresponding values of the variables are found on a line crossing the scales.

By the expression "corresponding values" is meant a set of values which satisfy the equation represented by the nomograph; and the crossing line, therefore, is properly termed an isopleth.

The aim is to substitute the simplest possible mechanical operation for the labor of solving the formula by computation; and, with this in view, the nomograph should preferably be designed so that its isopleths are rectilinear.

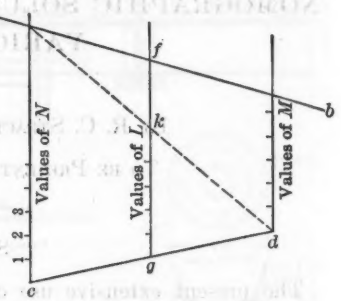

FIG. 1.

Take, as a simple illustration, three parallel scales of equal parts, starting from a common base line, $c\ d$, Fig. 1, and suppose the L line to be equidistant from the other two. Draw any straight line, $a\ b$, across the scales. Then $f\ g = f\ k + k\ g = \frac{1}{2} M + \frac{1}{2} N$.

The figure is a nomograph of the expression $L = \frac{1}{2}(M + N)$ and the line, $a\ b$, is an isopleth, its intersection showing by the scale readings a set of values which satisfy the equation.

If $g\ d = \frac{1}{3} c\ d$, $f\ g = \frac{1}{3} N + \frac{2}{3} M$, and the nomograph represents $L = \frac{N + 2\ M}{3}$, or $3\ L = N + 2\ M$, or $N = 3\ L - 2\ M$.

If the units on the L scale are made one-half as great as those on the other scales, the nomograph represents $L = \frac{2\ N + 4\ M}{3}$, or $3\ L = 2\ N + 4\ M$.

As another illustration, in Fig. 2, let $c\ d = d\ e$, and lay off the parallel scales for P and Q as shown. This is a nomograph of $P = \frac{1}{2} Q$, and any straight line drawn from c through the scales is an isopleth.

Evidently, changes in the positions and magnitudes of the scales may also be made in this diagram, with corresponding effects on the expression represented.

If in place of the numbers we lay off their logarithms, we may read products instead of sums; exponents may be affected by changing the distance between scales, and numerical coefficients may be introduced by varying longitudinally the position of one or more scales.

A straight-edge may be used to indicate the position of an isopleth, or, for greater ease in reading the scales, a strip of transparent celluloid having a line scratched on it, will be found very convenient.

The mathematical principles involved are very simple, and their application is easy when the right clue has been discovered. Many expressions which in their usual forms appear to be hopeless can be adapted to nomographic representation by a proper transformation; but the necessary change may not suggest itself until the equation has been studied somewhat thoroughly.

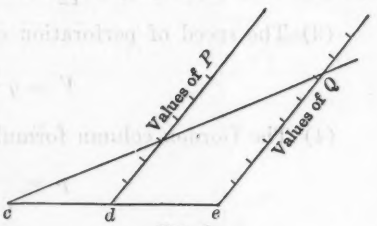

FIG. 2.

It will be found that, as a rule, scales of natural numbers are suitable for formulas in which the signs + and — appear; and logarithmic scales are adapted to those in which quantities affected by exponents must be multiplied or divided. In many cases, however, a formula in which there is an integral exponent may be transformed so as to permit the use of natural numbers for its nomographic scales.

Great ingenuity has been exhibited by various writers in constructing charts for formulas of much complexity: the work of Professor Maurice d'Ocagne is perhaps the most exhaustive, though both Soreau and Seco have shown that this field possesses astonishing possibilities.

Quite a large number of contributed articles on the subject have been published,* those of Professor John B. Peddle having been put into book form. Articles have also appeared by B. D. Dean, M. Am. Soc. C. E., and Mr. Richard Muller.†

At intervals, charts for complicated formulas have been published without explanation, except as to the method of using them; and though

* In the *American Machinist.*
† *Engineering News*, September 3d, 1908.

they are valuable to those who are satisfied as to their correctness, they are virtually empirical when used blindly.

Among earlier engineers, Messrs. Ganguillet and Kutter charted their well-known formula, and important additions to the chart were made afterward by Rudolph Hering, M. Am. Soc. C. E., who embodied his results in a paper* entitled "The Flow of Water in Small Channels."

A most interesting study of nomography was made by Mr. Rodolphe Soreau.† In addition to a new form of chart for Kutter's formula, Soreau has constructed nomographs for the solution of the following:

(1) The weir formula, in metric units,

$$Q = 95\ (H + 0.7)^{\frac{3}{2}};$$

(2) The volume of a conical frustum,

$$V = \frac{\pi\, h}{12}\ (D^2 + D\, d + d^2);$$

(3) The speed of perforation of a projectile,

$$V = q\, \frac{a^{0.75}}{p^{0.5}}\, t^{0.7};$$

(4) The Gordon column formula, expressed in metric units,

$$p = \frac{s}{1 + \dfrac{l^2}{n\, r^2}};$$

(5) The formula for the thickness of hollow cylinders,

$$m = \sqrt{\frac{R + p}{R - p}} - 1;$$

(6) The barometric formula for determining heights,

$$N = 18\,293\ (1 + 0.002837\ \text{cos. } 2\ \lambda) \left[1 + \frac{2\ (T + t)}{1\,000}\right] \text{log. } \frac{H}{h};$$

any many others.

The diagrams in common use almost invariably take the form of the locus of an equation referred to rectangular co-ordinates; and, in order to make them cover the required range, it is frequently necessary to use a great number of lines. This makes the diagram confusing and difficult to read, thus introducing chances for error. In case any of the factors are affected by exponents, the disadvantages of diagrams of this form are greatly increased.

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* *Transactions*, Am. Soc. C. E., Vol. VIII, p. 1.

† The results are found in the Memoirs of the Society of Civil Engineers of France for 1901.

The immense superiority of the nomograph is due to the following facts: that it is easily read with precision on account of having few lines; that it saves much valuable time, providing in effect a perfect tabulation of all possible values within the limits of the chart; and that it enables one to see instantly the effect of a change, whether small or great, in any of the variables. Furthermore, the solutions are equally simple, whichever quantity in the formula is unknown.

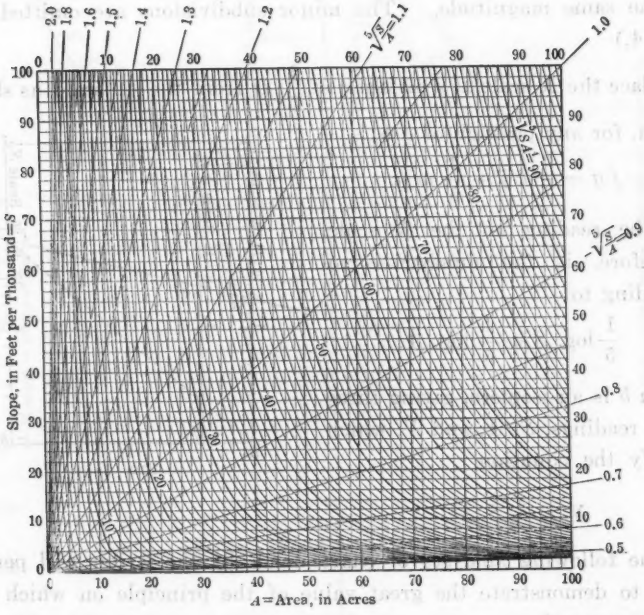


FIG. 3.

The following comparison will illustrate these points: In a recent issue of a technical periodical a correspondent gives the diagram, Fig. 3, for finding $\sqrt[5]{S\,A^4}$ in the McMath run-off formula.

He explains that, to construct this, it is necessary to compute $\sqrt[5]{\frac{S}{A}}$ and derive from this $\sqrt[5]{S\,A^4}$ for a number of cases sufficient to give all the points required for plotting the curves.

The finished diagram contains four sets of carefully plotted lines, and the intersections near the lower and left-hand edges are very indefinite.

A nomograph of this radical consists of three scales, and may be constructed with great ease and without computation, as follows:

Let $N = \sqrt[5]{S\,A^4}$.

Then $\log.\ N = \frac{1}{5}\log.\ S + \frac{4}{5}\log.\ A$.

Lay off three parallel scales of the logarithms of numbers from 1 to the limit desired (Fig. 4), making all three with logarithmic units of the same magnitude. (The minor subdivisions are omitted from Fig. 4.)

Place the N scale at a distance of $\frac{1}{5}\ D$ from the A scale as shown. Then, for any position of $a\ b$,

$$f\ g = \frac{1}{5}\,k\ l + \frac{4}{5}\,m\ n.$$

The reading on the N scale, therefore, is the number corresponding to

$$\frac{1}{5}\log.\ S + \frac{4}{5}\log.\ A,$$

and $a\ b$ is an isopleth, giving three scale readings, S, A, and N, which satisfy the equation,

$$N = \sqrt[5]{S\,A^4}.$$

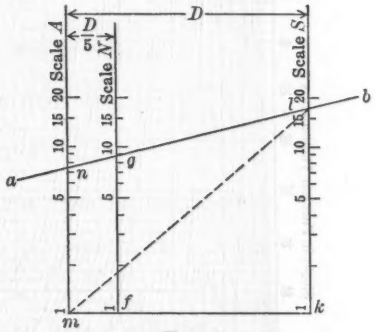


FIG. 4.

The following analyses of charts made by the writer will perhaps serve to demonstrate the great value of the principle on which they are constructed.

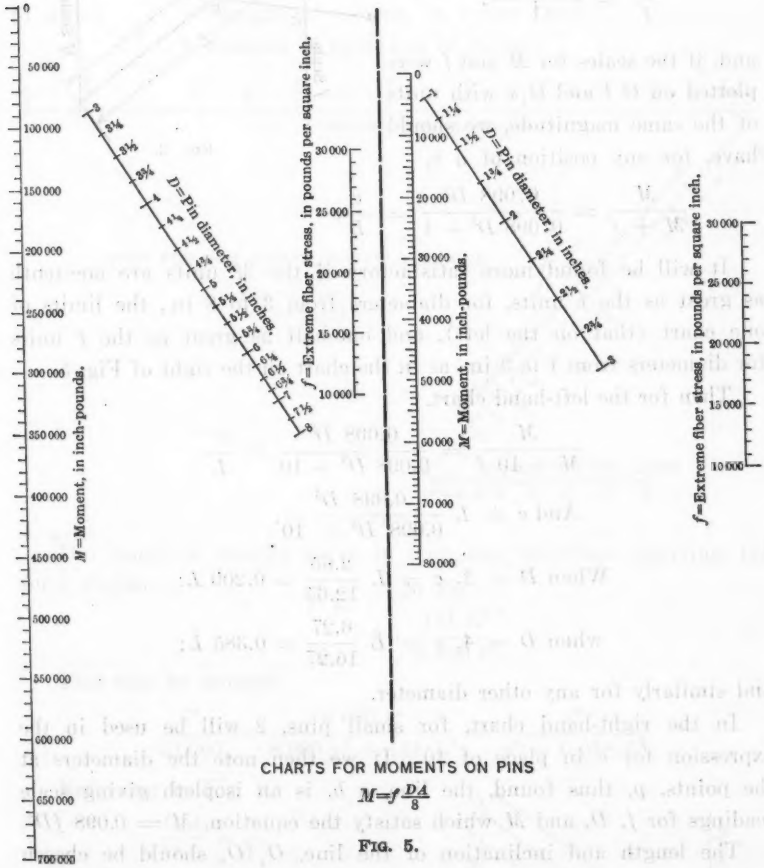
Chart for Pin Moments.—Tables for bending moments on pins are found in all engineers' pocketbooks, but interpolations are frequently necessary, particularly in reviewing a completed design. For this purpose a nomograph is especially useful, and a very neat one may be constructed with scales of natural numbers for moments and stresses. (Fig. 5.)

Let M = moment, in inch-pounds;
D = diameter of the pin, in inches;
A = area of the pin, in square inches;
f = extreme fiber stress, in pounds per square inch.

Then $M = f \frac{D A}{8}$.

Or, by substituting $0.7854\ D^2$ for A,

$$M = 0.098\ f\ D^3.$$



CHARTS FOR MOMENTS ON PINS

$M = f \frac{DA}{8}$

FIG. 5.

In Fig. 6, let the scales of equal parts for f and M be parallel, with the zeros at O and O_1, as shown, and plotted at a convenient distance apart. Connect O_1 and O, and let any line, $a\ b$, intersect the three lines.

Then $$\frac{O\,l}{O_1\,k} = \frac{c}{e},$$

therefore,

$$\frac{O\,l}{O\,l + O_1\,k} = \frac{c}{L}.$$

Also,

$$\frac{M}{f} = \frac{0.098\ D^3}{1},$$

and, if the scales for M and f were plotted on $O\ l$ and $O_1 k$ with units of the same magnitude, we should have, for any position of $a\ b$,

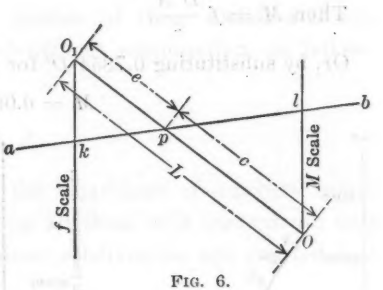


FIG. 6.

$$\frac{M}{M + f} = \frac{0.098\ D^3}{0.098\ D^3 + 1} = \frac{c}{L}.$$

It will be found more satisfactory if the M units are one-tenth as great as the f units, for diameters from 3 to 8 in., the limits of one chart (that on the left), and one-half as great as the f units for diameters from 1 to 3 in., as in the chart on the right of Fig. 5.

Then for the left-hand chart,

$$\frac{M}{M + 10\,f} = \frac{0.098\ D^3}{0.098\ D^3 + 10} = \frac{c}{L}.$$

$$\text{And } c = L\ \frac{0.098\ D^3}{0.098\ D^3 + 10}.$$

$$\text{When } D = 3,\ c = L\ \frac{2.65}{12.65} = 0.209\ L;$$

$$\text{when } D = 4,\ c = L\ \frac{6.27}{16.27} = 0.385\ L;$$

and similarly for any other diameter.

In the right-hand chart, for small pins, 2 will be used in the expression for c in place of 10. If we then note the diameters at the points, p, thus found, the line, $a\ b$, is an isopleth giving scale readings for f, D, and M which satisfy the equation, $M = 0.098\ fD^3$.

The length and inclination of the line, $O_1\ O$, should be chosen so as to give good intersections within the limits of the chart. A unit of 1 in. to 5 000 lb. for the f scale will be suitable for practical purposes.

Chart for Column Formula.—The Gordon column formula, when suitably transformed, may also be adapted to nomographic representation obtained by the relation of similar triangles.

The constants used in the chart, Fig. 7, are 20 000 and 8 000, or,

$$P = \frac{20\,000}{1 + \frac{l^2}{8\,000\,r^2}}$$

in which l = length of column, in inches; and
r = radius of gyration, in inches.

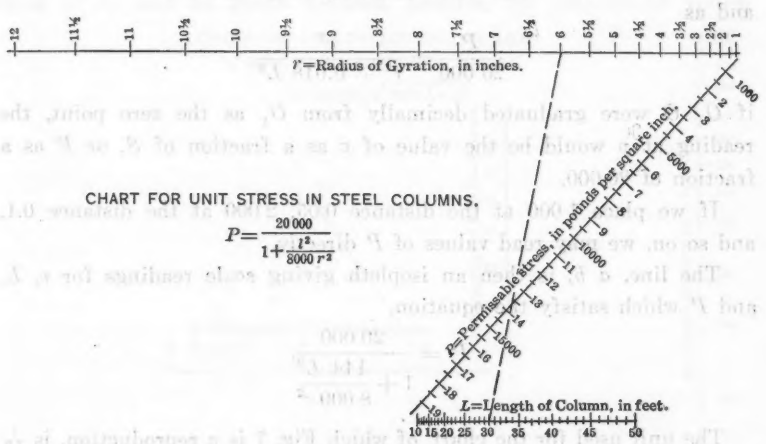


FIG. 7.

The length is usually given in feet, and therefore, inserting $12L$ for l, we have

$$P = \frac{20\,000}{1 + \frac{144\,L^2}{8\,000\,r^2}}.$$

This may be written

$$\frac{P}{20\,000} = \frac{8\,000\,r^2}{8\,000\,r^2 + 144\,L^2} = \frac{r^2}{r^2 + 0.018\,L^2}.$$

Referring to Fig. 8,

$$\frac{O_1 k}{O\,l} = \frac{c}{d}.$$

Therefore,

$$\frac{O_1 k}{O_1 k + O\,l} = \frac{c}{S}.$$

If now we lay off from O_1 1, 4, 9, 16, etc., units, placing the numbers 1, 2, 3, 4, at these points, and similarly from O 0.018 × 1,

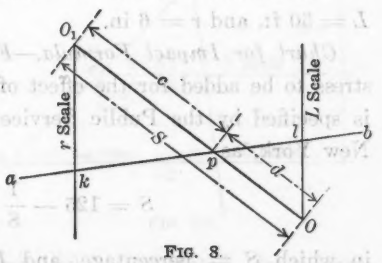


FIG. 8.

0.018×4, 0.018×9, 0.018×16 units, in other words, making the unit for the L scale 0.018 time the unit for the r scale, we shall have

$$\frac{r^2}{0.018\ L^2} = \frac{c}{d},$$

therefore,

$$\frac{r^2}{r^2 + 0.018\ L^2} = \frac{c}{S},$$

and as

$$\frac{P}{20\,000} = \frac{r^2}{r^2 + 0.018\ L^2},$$

if $O_1\ O$ were graduated decimally from O_1 as the zero point, the reading at p would be the value of c as a fraction of S, or P as a fraction of 20 000.

If we place 1 000 at the distance 0.05, 2 000 at the distance 0.1, and so on, we may read values of P directly.

The line, $a\ b$, is then an isopleth giving scale readings for r, L, and P which satisfy the equation,

$$P = \frac{20\,000}{1 + \frac{144\ L^2}{8\,000\ r^2}}.$$

The unit used for the chart, of which Fig. 7 is a reproduction, is $\frac{1}{10}$ in. for the r scale, and the distance, $O_1\ O$, is 10 in. These dimensions, of course, must be chosen with regard to the degree of precision desired in the scale readings. When constructed as stated, the value of P may be read to 100 lb.

Good intersections may always be obtained, since P is the same for $\frac{L}{r}$ and $\frac{n\ L}{n\ r}$.

The broken line on the chart shows the solution for the case $L = 30$ ft. and $r = 6$ in.

Chart for Impact Formula.—Fig. 9.—The percentage of live-load stress to be added for the effect of impact in designing bridge trusses is specified by the Public Service Commission of the First District, New York, as

$$S = 125 - \frac{1}{8}\sqrt{2\,000\ L - L^2}$$

in which S = percentage, and L = length of load producing the stress to which the percentage, S, is to be added.

If we represent $2\,000\ L - L^2$ by q, this may be written

$$125 - S = \frac{1}{8}\sqrt{q}.$$

Also, $$\log.\ (125 - S) = \frac{1}{2}\log.\ q - \log.\ 8.$$

The series of values of q, taken with suitable intervals in the value of L, may be easily deduced, because, for consecutive values

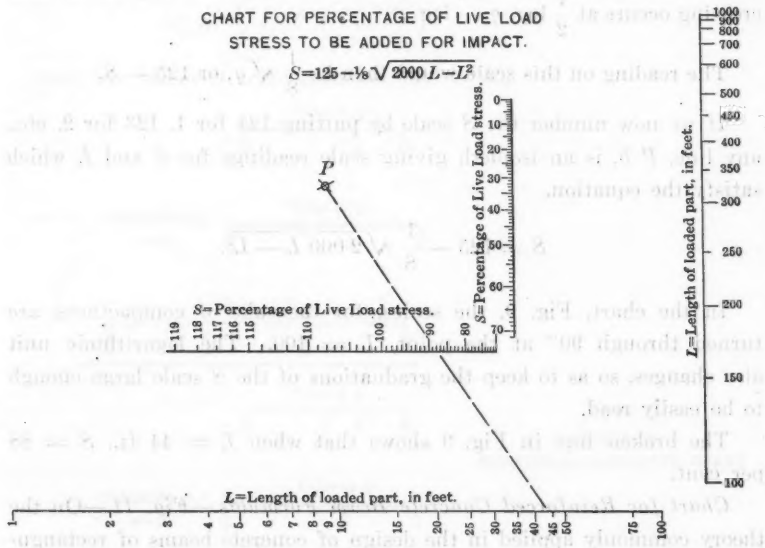


FIG. 9.

of L differing by unity, the consecutive values of q differ by $(2\,000 - 3)$, $(2\,000 - 5)$, $(2\,000 - 7)$, etc.

When $L = 1$,	$q = 1999$	$\log. q = 3.301$
2	3996	3.601
3	5991	3.777
4	7984	3.902
5	9975	3.999

Referring to Fig. 10, lay off the logarithms of q from the point, f, marking 1 opposite log. 1999, 2 opposite log. 3996, etc.

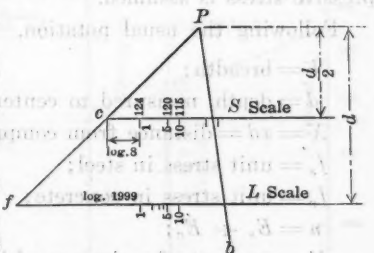


FIG. 10.

From the extremities of this scale, when completed, draw lines to a point, P, and place the line, S, parallel to the line, L, at a dis-

tance of $\frac{1}{2}d$ from it. If a logarithmic scale of the same unit magnitude as the scale, L, were placed on the line, S, beginning at c, any straight line from P through the L scale would give the number corresponding to $\frac{1}{2}$ log. q at the point of crossing the line, S; but, by beginning at a distance of 0.903 = log. 8 from c, the point of crossing occurs at $\frac{1}{2}$ log. q — log. 8.

The reading on this scale would then be $\frac{1}{8}\sqrt{q}$, or $125 - S$.

If we now number the S scale by putting 124 for 1, 123 for 2, etc., any line, $P\ b$, is an isopleth giving scale readings for S and L which satisfy the equation,

$$S = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2}.$$

In the chart, Fig. 9, the scales, for the sake of compactness, are turned through 90° at the point, $L = 100$. The logarithmic unit also changes, so as to keep the graduations of the S scale large enough to be easily read.

The broken line in Fig. 9 shows that when $L = 44$ ft., $S = 88$ per cent.

Chart for Reinforced Concrete Beam Formulas.—Fig. 11.—On the theory commonly applied in the design of concrete beams of rectangular cross-section reinforced on the tension side, the tensile strength of the concrete is neglected and a straight-line variation in compressive stress is assumed.

Following the usual notation,

b = breadth;
d = depth, measured to center of reinforcement;
$X = xd$ = distance from compression side to neutral axis;
f_s = unit stress in steel;
f_c = unit stress in concrete;
$n = E_s \div E_c$;
M = moment of resistance of beam;
p = percentage of reinforcement;
a = area of reinforcement per unit width = $p\ d$.

Then $$X = x\,d = d\left(\sqrt{2\,p\,n + p^2 n^2} - p\,n\right),$$

$$a = p\,d = \frac{d}{2\left(\frac{f_s}{f_c} + \frac{f_s^2}{n\,f_c^2}\right)},$$

$$M_s = f_s\,p\,d^2\left(1 - \frac{x}{3}\right),$$

$$M_c = \frac{1}{2}\,f_c\,d^2\left(x - \frac{x^2}{3}\right).$$

And, for balanced reinforcement, $M_s = M_c$.

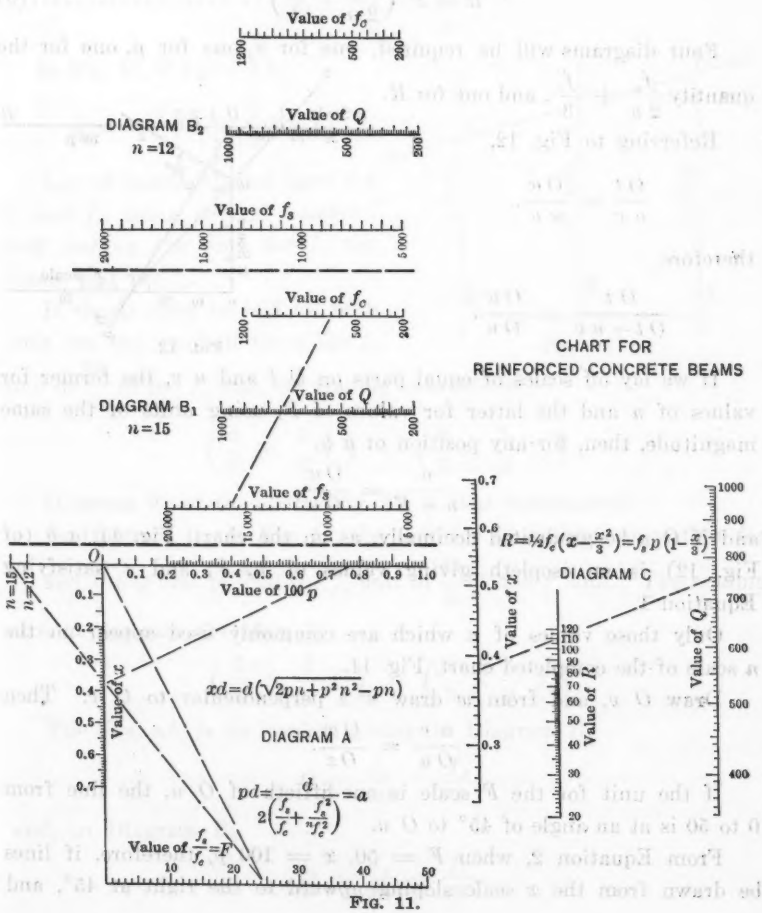


FIG. 11.

tance of $\frac{1}{2}d$ from it. If a logarithmic scale of the same unit magnitude as the scale, L, were placed on the line, S, beginning at c, any straight line from P through the L scale would give the number corresponding to $\frac{1}{2}$ log. q at the point of crossing the line, S; but, by beginning at a distance of 0.908 = log. 8 from c, the point of crossing occurs at $\frac{1}{2}$ log. q — log. 8.

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a = area of reinforcement per unit width = $p\,d$.

Then $$X = x\,d = d\left(\sqrt{2\,p\,n + p^2 n^2} - p\,n\right),$$

$$a = p\,d = \frac{d}{2\left(\frac{f_s}{f_c} + \frac{f_s^2}{n\,f_c^2}\right)},$$

$$M_s = f_s\,p\,d^2\left(1 - \frac{x}{3}\right),$$

$$M_c = \frac{1}{2}\,f_c\,d^2\left(x - \frac{x^2}{3}\right).$$

And, for balanced reinforcement, $M_s = M_c$.

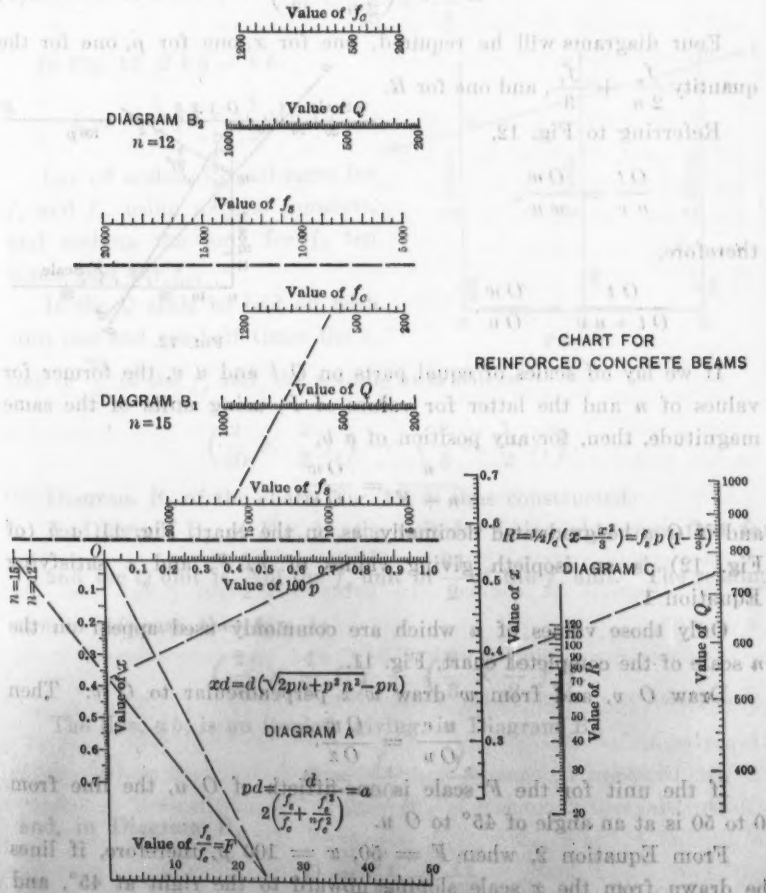

FIG. 11.

These expressions may be transformed so as to adapt them to nomographic representation.

If $$\frac{f_s}{f_c} = F,$$

and the moment of resistance $R\ b\ d^2$, or $R\ d^2$ per unit width,

Then $$x = \frac{n}{n+F} \quad \ldots\ldots(1)$$

$$p = \frac{x}{2F} \quad \ldots\ldots(2)$$

$$R = x^2\left(\frac{f_s}{2n} + \frac{f_c}{3}\right) \quad \ldots\ldots(3)$$

Four diagrams will be required, one for x, one for p, one for the quantity $\frac{f_s}{2n} + \frac{f_c}{3}$, and one for R.

Referring to Fig. 12,

$$\frac{O\,t}{u\,v} = \frac{O\,w}{w\,u},$$

therefore,

$$\frac{O\,t}{O\,t + u\,v} = \frac{O\,w}{O\,u}.$$

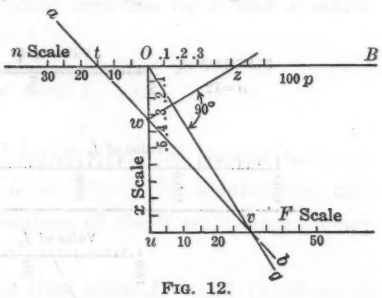

FIG. 12.

If we lay off scales of equal parts on $O\ t$ and $u\ v$, the former for values of n and the latter for values of F, using units of the same magnitude, then, for any position of $a\ b$,

$$\frac{n}{n+F} = \frac{O\,w}{O\,u},$$

and if $O\ u$ be graduated decimally, as on the chart, Fig. 11, $a\ b$ (of Fig. 12) is an isopleth giving values of n, F, and x satisfying Equation 1.

Only those values of n which are commonly used appear on the n scale of the completed chart, Fig. 11.

Draw $O\ v$, and from w draw $w\ z$ perpendicular to $O\ v$. Then

$$\frac{u\,v}{O\,u} = \frac{O\,w}{O\,z}.$$

If the unit for the F scale is one-fiftieth of $O\ u$, the line from 0 to 50 is at an angle of 45° to $O\ u$.

From Equation 2, when $F = 50$, $x = 100\ p$, therefore, if lines be drawn from the x scale sloping upward to the right at 45°, and

if the intersections with $O\ B$ are marked with the same figures as the points on the x scale from which they start, we shall have the scale of 100 p on $O\ B$.

A line, $O\ g$, through the F scale and a line perpendicular to $O\ g$ through w and $O\ B$ will then give values of F, x, and p, satisfying Equation 2. This completes Diagram A of the chart, Fig. 11.

To obtain R it is necessary first to obtain $\frac{f_s}{2\ n} + \frac{f_c}{3}$. Denoting this quantity by Q, when $n = 15$,

$$Q = \frac{f_s}{30} + \frac{f_c}{3}.$$

In Fig. 13, if $k\ g = k\ h$,

$$k\ m = \frac{1}{2}\ g\ n + \frac{1}{2}\ h\ p.$$

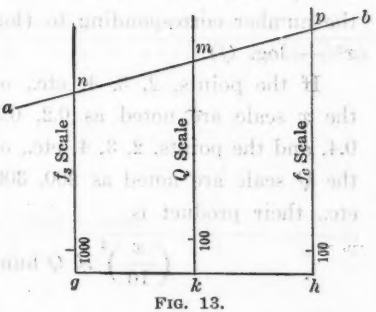

FIG. 13.

Lay off scales of equal parts for f_s and f_c, using natural numbers, and making the unit for f_c ten times that for f_s.

If the Q scale be laid off with unit one and one-half times the f_c unit or $\frac{30}{2}$ of the f_s unit, the reading at m will be

$$\left(\frac{2}{30} \times \frac{1}{2}\ f_s\right) + \left(\frac{2}{3} \times \frac{1}{2}\ f_c\right).$$

Diagram B_1 of the chart, Fig. 11, is thus constructed.

In Diagram B_2, of Fig. 11, the unit for f_c is eight times that for f_s, and the Q unit is $\frac{3}{2}$ of the f_c unit or $\frac{24}{2}$ of the f_s unit. The reading at m in this case, therefore, is

$$\left(\frac{2}{24} \times \frac{1}{2}\ f_s\right) + \left(\frac{2}{3} \times \frac{1}{2}\ f_c\right).$$

The line, $a\ b$, is an isopleth giving, in Diagram B_1,

$$Q = \frac{f_s}{30} + \frac{f_c}{3},$$

and, in Diagram B_2,

$$Q = \frac{f_s}{24} + \frac{f_c}{3}.$$

Diagram C, of Fig. 11, is constructed to give the product $x^2Q = R$, as follows:

Place the R line as shown in Fig. 14, making $k\,g = \frac{1}{3}\,g\,h$. Lay off the logarithms of x and Q on their respective lines, using the same unit. Then, for any position of $a\,b$,

$$k\,m = \frac{2}{3}\log.\ x + \frac{1}{3}\log.\ Q = \frac{1}{3}(\log.\ x^2 + \log.\ Q).$$

The R scale is laid off with unit one-third of that used for x and Q, making the reading at m the number corresponding to (log. x^2 + log. Q).

If the points, 2, 3, 4, etc., on the x scale are noted as 0.2, 0.3, 0.4, and the points, 2, 3, 4, etc., on the Q scale are noted as 200, 300, etc., their product is

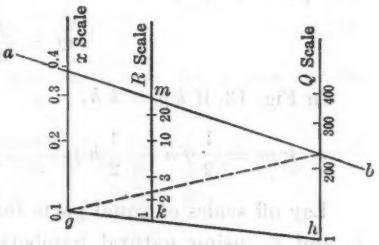

FIG. 14.

$$\left(\frac{x}{10}\right)^2 \times Q \text{ hundreds} = x^2\,Q \text{ units}.$$

Diagram C is then a nomograph for the expression, $R = x^2Q$, Q being taken from Diagram B_1 when $n = 15$, and from Diagram B_2 when $n = 12$, f_c and f_s being in pounds per square inch, b and d in inches, and $R\,b\,d^2$ in inch-pounds.

To illustrate the use of the chart, Fig. 11, assume $f_s = 16\,000$, $f_c = 650$, $n = 15$, and $b = 12$ in.

Moment of external forces = 517 000 in-lb.

On Diagram A connect the point $n\,15$ with $F\,24.6$ and read $x = 0.38$.

Connect the origin, O, with $F\,24.6$, and, on a perpendicular to this line, drawn from 0.38 on the x scale, read $100\,p = 0.77$, or $p = \frac{77}{100}$ of 1 per cent.

On Diagram B_1 connect f_s 16 000 with f_c 650, and read $Q = 750$.

On Diagram C connect x 0.38 with Q 750, and read $R = 109$.

Then $517\,000 = 109 \times 12\ d^2$,

therefore, $d = 20$ in.

Allowing 2 in. of concrete below the center of reinforcement, the required beam would then be 12 by 22 in., and the steel area would be 0.77% of 240 sq. in. = 1.85 sq. in.

Many hydraulic formulas contain fractional exponents, and are cumbersome on account of the time required for their solution. This difficulty is partly overcome by tabulating the results for a series of cases. Interpolations in such tables, however, are not invariably

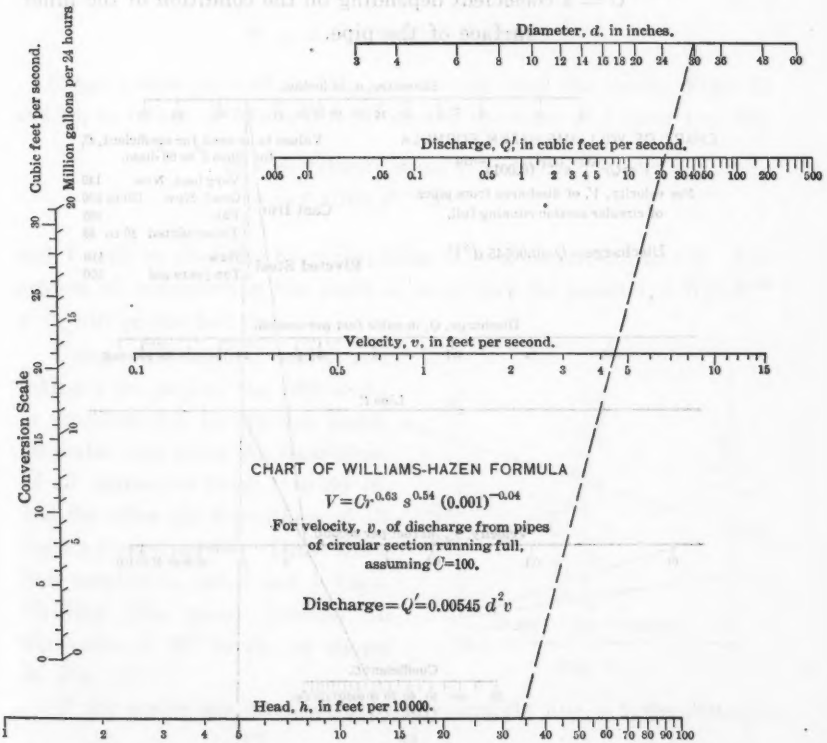

FIG. 15.

simple, and resort is had to many expedients to simplify the work. Special "hydraulic slide-rules" have been devised, which give, with sufficient precision, the results of the formula for which they are designed, but naturally such special rules can be of no general utility.

The charts for the Williams-Hazen formula, Figs. 15 and 16, are given as examples of the beauty of the nomograph in its application to the solution of an exponential equation.

Charts for Williams-Hazen Formula.—As usually stated, this formula is

$$V = Cr^{0.63}S^{0.54}0.001^{-0.04},$$

in which V = velocity, in feet per second;
r = mean hydraulic radius;
S = slope or ratio of rise to length;
C = a coefficient depending on the condition of the inner surface of the pipe.

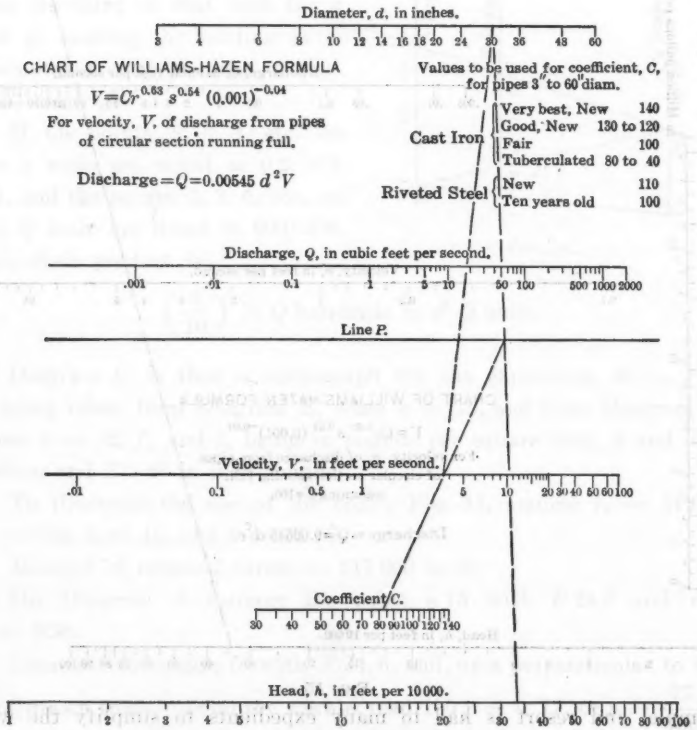

FIG. 16.

In constructing a nomograph of this formula, applicable to pipes of circular section running full, it will be convenient first to transform it so as to express r in terms of the diameter, and s as feet of head in 10 000 ft.

Letting d = diameter of pipe, in inches;
and h = feet of head in 10 000 ft.,

then $$r = \frac{D\text{ (feet)}}{4} = \frac{d\text{ (inches)}}{48},$$

and $$S = \frac{h}{10\,000}.$$

Making these substitutions, we obtain

$$V = 0.000796\ C\ d^{0.63}\ h^{0.54}.$$

C has values from 30 to 140, and we will limit the charts, Figs. 15 and 16, to values of d from 3 to 60 in. and to values of h from 1 to 100.

If v = velocity when $C = 100$,

$$v = 0.0796\ d^{0.63}\ h^{0.54},$$

and V will be obtained by multiplying v by a factor, 0.3 to 1.4. The process of constructing the chart so as to give the product, $0.0796\ d^{0.63}\ h^{0.54}$, will be the first step.

Using the decimal scale, and taking ½ in., or 1 on the 20th scale, to represent 0.1, lay off two parallel scales, one being the logarithms of all diameters from 1 to 60 in. and the other the logarithms of all the h's from 1 to 100. Draw the v line parallel to the d and h lines, dividing the space between in the ratio of 63 to 54, as shown in Fig. 17.

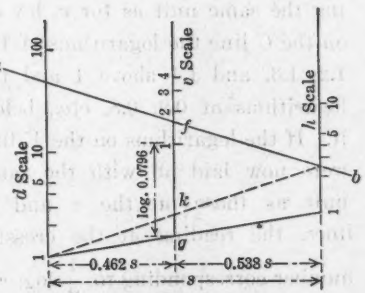

FIG. 17.

If the scales are intersected by any straight line, $a\ b$, the distance, $f\ g = f\ k + k\ g = \frac{63}{117}\log.\ d + \frac{54}{117}\log.\ h$. If the v's were laid off by the scale of 20ths beginning at the point, g, the reading at f would give the product, $d^{\frac{63}{117}}\ h^{\frac{54}{117}}$.

$\frac{63}{117} = 0.855 \times \frac{63}{100}$, hence if the logarithms of the v's are laid off with a unit of (½ in. × 0.855) to 0.1, the reading at f would give the

product, $d^{0.63}\ h^{0.54}$. This may be accomplished by using for the v's the scale of 20ths after multiplying the logarithms by 0.855, thus:

$$\text{log. } 2 = 0.301 \times 0.855 = 0.257$$
$$\text{log. } 3 = 0.477 \times 0.855 = 0.408$$
$$\text{log. } 4 = 0.602 \times 0.855 = 0.515,$$

or by any other convenient method of reducing the unit magnitude.

The logarithm of $0.0796 = \bar{2}.901 = -1.099$, and by placing the starting point of the v's above g a distance $1.099 \times 0.855 = 0.940$ on the scale of 20ths, thus adding -1.099, the logarithm of 0.0796, the reading on the v line is $0.0796\ d^{0.63}\ h^{0.54}$.

To introduce the factor, C, draw the C line parallel to the others at any suitable place between the v line and the h line. Draw the V line midway between the v line and the C line, as in Fig. 18.

From the point, 1, on the v scale, draw a line across the v, V, and C lines at any convenient angle, in order to locate the "1" points of the scales about to be plotted. Using the same unit as for v, lay off on the C line the logarithms of 1.1, 1.2, 1.3, and 1.4 above 1 and the logarithms of 0.9, 0.8, etc., below it. If the logarithms on the V line were now laid off with the same unit as those on the v and C lines, the reading at the crossing of any line, $a\ b$, would be the number corresponding to $\frac{1}{2}$ log. $v + \frac{1}{2}$ log. C.

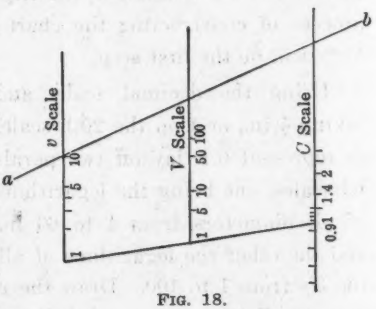

FIG. 18.

Therefore we make the V scale one-half as large as the other two, that is, with a unit of ($\frac{1}{4}$ in. $\times$ 0.855), which will give the product, $v\,C$, as required.

It is apparent that no divisions are required on the v line after the starting points for the V and C scales are drawn. Therefore it is left blank in the completed chart, Fig. 16, and marked "Line P" for reference.

In order to make the chart of greater use, it will be well to introduce another scale giving Q, the quantity discharged.

If Q = discharge, in cubic feet per second;
d = diameter, in inches;
V = velocity, in feet per second;
$Q = 0.00545 d^2 V$.

The d scale was plotted with a unit of $\frac{1}{2}$ in. to 0.1. Therefore it will serve for log. (d^2) with a unit of $\frac{1}{4}$ in. to 0.1. The V scale is made with a unit of ($\frac{1}{4}$ in. $\times$ 0.855), as shown on Fig. 19.

If we place the Q line between the d and V lines, as shown, the distance, $f\,g = n \times$ log. (d^2) + (1 — n) $\times$ 0.855 log. V, and if $n = (1 - n) \times 0.855$,

$$n = 0.462$$

Then $$f\,g = 0.462 \text{ log. } (d^2) + 0.462 \text{ log. } V.$$

If the Q scale were now laid off from g with a unit of ($\frac{1}{4}$ in. $\times$ 0.462), the reading at f would be the number corresponding to log. d^2 + log. V.

Log. 0.00545 = $\bar{3}.7364$ = —2.2636, and if we place the "1" of the Q scale above g a distance, $2.2636 \times 0.462 = 1.045$ on the 40th scale ($\frac{1}{4}$ in. representing 0.1), thus adding log. 0.00545, the reading at the intersection of a b will give $0.00545\ d^2\ V$, as required.

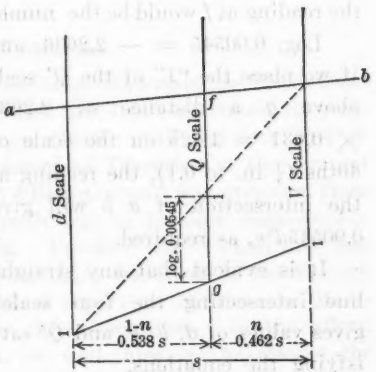

FIG. 19.

Assuming d and h to be known, and a value of C selected according to the condition of the pipe, place a straight-edge so as to connect d and h, and mark its intersection on "Line P". Similarly, connect the point just marked with the given C and at the intersection on the V line read the value of V. Then connect this V with the given d, and at the intersection on the Q line read Q in cubic feet per second.

It is evident that, by a suitable modification of the process described, either of the four quantities, d, h, V, or C, may be obtained when the other three are given.

As C is usually assumed at a round figure, multiplication or division by it is so simple that some might prefer a chart based on

the value 100 for this coefficient. The chart, Fig. 15, is constructed in this way, as follows:

Lay off the d, h, and v scales as described. The v scale will have its subdivisions completed and will occupy the position of "Line P" in the chart, Fig. 16.

The subdivisions on the d line represent log. (d^2) with a unit of $\frac{1}{4}$ in., and those on the v line represent log. v with a unit of ($\frac{1}{2}$ in. $\times$ 0.855), equivalent to $\frac{1}{4}$ in. $\times$ 1.71.

If we place the Q' line as shown in Fig. 20, the distance,

$$f\,g = m \text{ log. } (d^2) + (1 - m) \times 1.71 \text{ log. } v;$$

and if $m = (1 - m) \times 1.71$, $m = 0.631$.

Then $$f\,g = 0.631 \text{ log. } (d^2) + 0.631 \text{ log. } v.$$

If the Q' scale were laid off from g with a unit of ($\frac{1}{4}$ in. $\times$ 0.631), the reading at f would be the number corresponding to log. (d^2) + log. v.

Log. 0.00545 = — 2.2636, and if we place the "1" of the Q' scale above g a distance of 2.2636 $\times$ 0.631 = 1.428 on the scale of 40ths ($\frac{1}{4}$ in. to 0.1), the reading at the intersection of a b will give $0.00545d^2v$, as required.

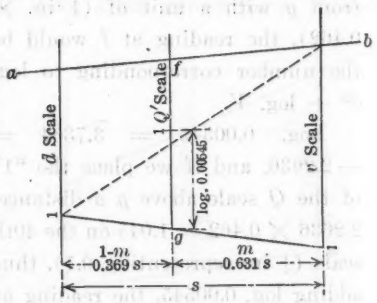

FIG. 20.

It is evident that any straight line intersecting the four scales gives values of d, h, v, and Q' satisfying the equations,

$$v = 100 \times 0.000796\, d^{0.63} h^{0.54},$$

and $$Q' = 0.00545 d^2 v.$$

By means of the "conversion scale", cubic feet per second may be read off at once, in millions of gallons per day.

The broken lines drawn on the charts, Figs. 15 and 16, show that when $d = 30$ in., $h = 33$ ft., and $C = 82$, the formula gives $V = 3.7$ and $Q = 18$ cu. ft. per sec. Also when $d = 30$ in. and $h = 33$ ft., $v = 4.5$ and $Q' = 22$ cu. ft. per sec.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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PROGRESS REPORT OF THE SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE.*

To the American Society of Civil Engineers,

Your Special Committee on "Bituminous Materials for Road Construction, and on Standards for Their Test and Use", continued its work as such since its report of a year ago (1913) until the first part of June, 1914, when, by action of the Board of Direction at the Annual Convention of the Society, the word "Bituminous" was stricken from the title of the Committee. The result was to broaden considerably the field of the Committee's work. Further, the Committee has become aware of an expectation among many members of the Society that something in the nature of a final report on Bituminous Materials would shortly be presented by this Committee. In the effort, therefore, to meet this expectation, as well, also, as to release the Committee for its work in the broadened field of its action, the attempt is now made to present a comprehensive report, and one as nearly final as may be, under all the circumstances, on "Bituminous Materials Used in Road Construction, and on Standards for Their Test and Use".

It should be recognized that many points in reference to these materials are as yet unsettled, and are improbable of early settlement, because of the further investigations necessary to determine certain facts, or because of the insufficient time that has elapsed since certain investigations were begun in order to establish the facts. Under the existing arrangements, and in the manner proposed by the Committee, the latter will report from time to time any additional conclusions it may reach, on this particular subject, which it believes to be of value to the Society.

* To be Presented to the Annual Meeting, January 20th, 1915.

Since the publication of the Report of the Committee for 1913, some criticisms of it, or of the expressions used, have appeared. The Committee has considered these criticisms carefully, and has taken them into account in preparing this report.

Since June, 1914, the Committee has considered the scope of its work as your Special Committee on "Materials for Road Construction, and on Standards for Their Test and Use", but has concluded that a Semi-final Report on the "Bituminous Materials" should be prepared for presentation to the Society before it should undertake the investigations and collection of data necessary for a Progress Report on "Materials Other than Bituminous". The Committee therefore submits the following:

SEMI-FINAL REPORT ON BITUMINOUS MATERIALS USED IN ROAD-MAKING AND MAINTENANCE, WITH STANDARDS FOR THEIR TEST AND USE.

For more than 50 years, the use of Bituminous Materials of various kinds has prevailed in highway work to a greater or less extent in the United States, and for a longer period in some other countries. Except in the cases of a few materials, this use, until within 10 years or less, had been marked by the variety of materials used, the variety of methods of use, the variety of results secured, the lack of standards for materials, methods, and results, and the lack of appreciable advance in the science and art underlying the results.

In the exceptions referred to, which are meant to include the sheet-asphalt and certain bituminous concrete pavements, it may be said that progress had been steadily—if somewhat slowly—made in the sheet-asphalt work, and that the development of the bituminous concrete pavement rapidly advanced the science and art of bituminous pavements generally.

However, even including both these cases, as late as 6 years ago (1909) standards for materials were lamentably lacking. It was then customary, even where the specifications were otherwise all that could be expected, to describe the bituminous material by its trade name or as "equal to" a particular brand of fairly well-known character. Highway authorities generally knew little or nothing as regards means of identification of bituminous materials—even the asphalts coming under this statement—and no considerable agreement existed among engineers or chemists connected with highway work as to the analyses and tests desirable to be made on either the better known asphalts or on the newer materials offered for use. A suggestion then made, to the effect that bituminous materials could and should be purchased under specifications which would describe their essential characteristics, met with discouragement and even ridicule from many quarters. However, in 1909, your Board of Direction received and acted on

the suggestion that a Special Committee be appointed on "Bituminous Materials for Road Construction, and on Standards for Their Test and Use", and in May, 1909, appointed such a Committee.

It should be realized that, although European engineers have been constructing and maintaining bituminous pavements since the middle of the Nineteenth Century, your Board has authorized the first systematic and comprehensive work which has been undertaken in the endeavor to place the use of bituminous materials in highway work on a scientific basis.

The Committee, recognizing the diversity in both form and character of the available information on the subject, immediately set about collecting and co-ordinating such facts as might be obtained and securing additional information on uniform lines which seemed to it suitable. To this end, your Committee prepared blanks for the purpose of recording uniformly such exact information as apparently was desirable of collection from many sources, and distributed these forms widely, requesting that they be used for forwarding to the Committee the facts concerning the use of bituminous materials in construction. Later, the Committee prepared and distributed similar forms for data concerning the use of bituminous materials in maintenance work. The Committee regrets having to record a less generous response to its requests for information than has been hoped for. The Permanent International Association of Road Congresses printed in full the forms prepared by your Committee, and distributed them, in special pamphlet form, to the members of the Second International Road Congress held at Brussels, Belgium, in 1910. A complete revision of these forms has recently been made by the Committee, and the old and revised forms are attached to this report as Appendices A and B, respectively. The Analyses and Tests now recommended will be found in Appendix C.

The Committee made the effort to include in its schedules the points that seemed important to others of recognized experience in such matters; to adopt, as far as practicable, methods already standardized; and to make only such changes in ordinary methods as seemed to the Committee to be necessary for accuracy or improvement. To some extent, new methods, analyses, and tests, had to be suggested by the Committee, and the latter acted in these cases only after careful consideration and after conferring with many interested parties, including the manufacturers of such materials.

The investigations and correspondence of the Committee revealed such a variety of opinions concerning bituminous materials and their use in highway work that, not only was the action of your Board of Direction in creating the Special Committee supported, but also the Committee was moved to arrange with the Board of Direction for an opportunity to have such matters presented and discussed.

These arrangements resulted in special meetings of the Society being held just after the Annual Meetings of 1911, 1912, 1913, and 1914 (in 1910 an informal meeting under the auspices of the Committee was similarly held, but no special meeting of the Society itself, as in the later years). Through these meetings the interest of highway authorities was aroused and developed, and the papers that were presented, with the discussion thereon, undoubtedly did much to further the efforts of the Committee in its work as well as to advance the knowledge of highway matters generally.

In 1912, the Committee felt able to submit some conclusions unanimously agreed upon by its members. There were others that were strongly supported in the Committee, but which were withheld because of the apparent wisdom of acting conservatively in view of the rather small number voting. In this Report (January, 1912) the Committee also recommended that the attention of all interested be especially directed during the ensuing year to certain details of use, for the purpose of securing a consensus of opinion based on facts concerning these points, and that the definition of two terms of common use be adopted.

In 1913, the Committee again submitted a progress report, with some conclusions on various points, and recommended the adoption of the definition of another term of growing general use.

In 1914 (under date of November 15th, 1913) the Committee submitted a report of progress, which also summarized its earlier conclusions and collected all the latter in one place for the sake of easy reference and convenience. In this Report were included a number of new conclusions of the Committee (or original expressions of conclusions not before expressed though perhaps quite generally understood and accepted), and more than one hundred terms with their definitions, as recommended for adoption by the Committee.

As previously referred to, and for various reasons, it has seemed wise to the Committee to act conservatively in presenting conclusions to the Society. As your Board of Direction, however, early in 1914, added three new members to the original Committee of four, it has been possible to accomplish more results during 1914, especially in view of the ripening fruit of the earlier work of the original Committee, and it seems probable that such will also be the case for the near future at least.

At the present time it may be said that a number of important questions concerning the use of bituminous materials in highway work are desirable of investigation or of solution. Some investigations have been begun, but, as yet, have not been concluded successfully mainly because of lack of sufficient time for the proof of certain principles. Other investigations should be begun and pursued under such a variety of conditions as will render the conclusions of general

value. As the Committee has before stated (in its Report dated January, 1912), it believes "that, in order to solve many of the problems of construction and maintenance, it is more necessary to have at hand physical data recorded along uniform lines for a relatively short period than to prove the correctness of certain fine theoretical points", and, therefore, it again urges resistance to a tendency, apparent in some instances, to ignore the opportunity offered by the Committee for collaboration, which would result in mutual benefit, and deprecates any tendency to return to the former chaotic conditions which would ensue from individual, non-concurrent effort.

Of the investigations above mentioned as begun, but as yet unfinished, the following may be mentioned as important and worthy of conclusion:

(1) The effects of various kinds of earth or soil on bituminous carpets under known conditions of traffic;
(2) The efficiency of the use of asphaltic oils for surface treatments on gravel and broken stone roadways;
(3) The desirable analyses and tests to be made on bituminous materials for known conditions of use;
(4) The standardization of a determination and expression of the consistency of bituminous materials which preferably may permit of its use on all the wide range of such materials, from liquid to solid, and of the inter-relation existing in certain, if not in all, cases;
(5) The causes of the formation of waves or corrugations in bituminous surfaces and pavements;
(6) The use of a volatile flux instead of the application of heat for the purpose of rendering the bituminous material or cement sufficiently fluid for use under any method;
(7) The maximum quantity of distillate coming over, up to 170° cent., from tars (as determined by the test proposed by the Committee) which may be properly allowed in a material under any known conditions of use and of traffic;
(8) The maximum and minimum quantities of free carbon that can be successfully allowed in tars under known conditions of other constituents of the tar, of climate, of traffic, and of methods of use;
(9) The advisability of incorporating in bituminous pavements, built by the penetration method, bituminous material in excess of the minimum quantity necessary for the production of satisfactory results;
(10) The relative efficiency of the various pressures used in the application of various grades and types of bituminous materials in the construction of bituminous surfaces and bituminous pavements built by the penetration method;

(11) The variable allowable limits of the percentage of bitumen in bituminous concrete mixtures, dependent on the kind and sizes of the component parts of the mineral aggregates;

(12) The allowable maximum penetrations of various types of asphalt cements when such cements are used in bituminous concrete pavements which must be open to traffic shortly after being laid;

(13) The efficiency of bituminous concrete pavements constructed with a number of courses composed of different sizes of broken stone;

(14) The allowable minimum atmospheric temperatures under which it is practicable to secure the desired compressions of bituminous concrete mixtures composed of various grades of different types of bituminous cements and different kinds of mineral aggregates; and

(15) The use of "seal coats" on bituminous pavements under stated conditions.

Of the investigations before mentioned as important and worthy of commencement and prosecution, the following may be noted:

(1) The establishment of standards for recording, compiling, and reporting costs of results secured in highway work;

(2) The establishment of standard forms of specifications for the various kinds of bituminous surfaces and pavements ordinarily used, including the bituminous materials therefor;

(3) The devising of standards for recording and compiling traffic censuses, especially to the end that comparisons of traffic can be made readily and with relative accuracy; and

(4) The establishment and expression of any relation between the traffic and the life or cost of any bituminous surface or pavement.

The Committee now submits its conclusions on Bituminous Materials with the new or the modified forms of its earlier reported conclusions in heavy type, as follows:

General.—The Committee deplores the tendency, apparent in some quarters, to devote time and energy for the discovery of "The most satisfactory road surface", and expresses its conviction that there is no such thing as a "panacea" for all highway ills. It believes that, with the development of highway work, it should be constantly more apparent that one of the greatest problems to be solved by highway engineers is the proper selection of the particular material and form of construction to be used, which most efficiently meet the conditions of any particular case, and that progress will be hastened by complete recognition of this fact.

Most materials and methods of use of bituminous materials have their values, and the real problems are the determination of these values and of their adaptability or fitness to meet the conditions of any case properly.

A bituminous surface or pavement properly designed for carrying light motor vehicles may not be the one best adapted to horse-drawn vehicles, and one which is efficient for light horse-drawn vehicles may be most seriously injured by motor vehicles. A solution of the problem of the proper design and construction of a roadway for mixed traffic is, under any local conditions on which the solution ultimately depends, difficult, though facts are accumulating which may finally permit this solution.

It is still desirable to record the characteristics and details (including cost figures) of use of bituminous materials, and great concordance in so doing is most necessary.

Census of Traffic.—Your Committee desires to emphasize the fact that experience has demonstrated the value of a traffic census taken both preliminary and subsequent to the construction of a highway.

The traffic census should be considered one of the most important variable factors in the solution of that important problem, the selection of that type of construction best suited to local conditions considered from the standpoints of both economy and efficiency. In connection with the census returns on any highway should be considered the traffic on cross and parallel highways and the effect of improvement of these highways on the traffic of the highway under construction.

It should not be taken for granted that the bald return of a traffic census should be the sole basis of the selection of the type of construction, but it should be considered a guide in determining the value of the type adopted.

Where motor traffic forms a considerable proportion of the total traffic likely to use a highway, the unit width of traffic lines should be considered as 9 or 10 ft., instead of 7 or 8 ft., as heretofore, because of the greater clearance required for the safe passing of the units of such traffic.

In considering the effect of traffic and its relation to the design and cost of maintenance, it is necessary to take into account the speed as well as the weight of the vehicles.

In view of the recent constant and rapid increase of traffic on the highways, both in number of vehicles and in wheel loads, it will be in the interests of economy for designs of highways to be made with proper consideration of further increases.

Collection and Standardization of Cost Data.—In the work of the Committee, it has become apparent that conclusions may necessitate the use of cost data, recorded along uniform lines. It seems to be

urgent that standards for arriving at costs be established and generally adopted as promptly as possible. Your Committee therefore recommends to the attention of the Profession this important feature of engineering work, with the hope that uniform methods will be generally adopted at an early date for reporting Costs of Construction and of Maintenance.

The Committee believes that, along the lines of standardization in reporting cost data and for general benefit, it would be advisable for highway departments, in making their regular periodic reports, to adopt a tabular form for presenting their records of the costs of their construction work, which should include data or figures in columns as follows:

Identification of Work;
Description (Width, Thickness, and Material or Form of Construction);
Length;
Area of Bituminous Pavement or Surface;

Construction Expenditures for
- **Preliminary surveys and plans;**
- **Grading;**
- **Surfacing;**
- **Bridges and culverts;**
- **Under-drains;**
- **Superintendence and inspection;**
- **Miscellaneous;**

Total of above;
Rights of Way and Damages;
Total Construction and Rights of Way and Damages;
Administration, Legal and General Engineering Expenses;
Total Expenditures and Overhead Expenses;

For the maintenance expenditures there should be a tabular form showing, in columns:

Identification of work;
Length;
Area;

Maintenance Expenditures for
- **Labor;**
- **Material;**
- **Hire of teams and equipment;**
- **Inspection and superintendence;**
- **Miscellaneous;**

Total of above;
Administration, Legal and General Engineering Expenses;
Total Cost of Maintenance.

Each table, of course, should carry such foot-notes or text as might be necessary to supply the further information as to unusual rates of pay or other conditions advisable to be shown.

In compiling costs of any item of work done, the Committee believes that it would be advisable and of great public benefit if, hereafter, all such figures be compiled whenever practicable on the following basis:

Total Cost to be Reported per Unit for
- **Cost for materials delivered on site of work;**
- **Cost for labor and teams placing materials;**
- **Cost for machinery engaged in placing materials and in finishing the work;**
- **Cost for inspection and superintendence;**
- **Cost for general overhead charges.**

In foot-notes or in the text such statements as may seem desirable in any case should be made concerning rates of pay, conditions, and circumstances.

Construction—General.—Your Committee is agreed that:

The use of any form of a bituminous pavement or bituminous surface does not preclude the necessity for the construction of a well-drained, thoroughly compacted, and adequate foundation. In fact, such improvement of the highway frequently attracts heavier traffic and thus increases the stresses on the sub-grade.

Within certain limits of traffic, the proper use of bituminous material should effect economy in the expenditures for the cleaning and maintenance.

The proper treatment of a broken stone, gravel, shell, or slag roadway with bituminous material, for the purpose of eliminating the so-called dust nuisance, will at the same time render even the best of such roadways more efficient for sustaining traffic, and such treatment with bituminous materials is usually preferable and more economical than sprinkling with water, or the use of hygroscopic salts.

An objectionable slipperiness of bituminous pavements or bituminous surfaces may be decreased or prevented by proper precautions during construction or by proper treatment thereafter.

The crown generally used in the construction of broken stone roadways is excessive when bituminous materials are used, and a crown of even 1/2 in. per ft. should be avoided when a lesser crown can be secured without detriment to the surface drainage.

Construction—Materials.—Your Committee is agreed that:

For the present, at least, whenever comprehensive specifications are to be prepared so as to admit a variety of types of bituminous materials, separate specifications as may be necessary should be prepared for each type.

Where bituminous pavements are laid, the edges should be protected and a sudden transition from the pavement to any softer shoulder material avoided by means of cement concrete or other

edgings and such reinforcement of the shoulder material as may be necessary.

The character of the stone to be used may influence the choice of the bituminous material.

Whatever method may be used in any case, it is essential, as in water-bound construction, that a suitable quality of road metal be used.

By proper selection and use of the bituminous material, injury to property and deleterious effects upon animal and vegetable life may be avoided, and also considerable hygienic advantage may result from the use of such materials on the highways.

The maximum quantity of distillate up to 170° cent. allowable in tars depends on the proportions of the other fractions in a tar, and on the conditions of use of the tar, and possibly on the conditions on the road surface after its construction, or on some other facts.

Construction—Methods.— **Your Committee wishes to emphasize the fact that the selection of the method of construction to be used in a given case will depend on the results of preliminary investigations, and that the recent and imminent development of, and the availability of, mechanical appliances, should affect the decision to a great degree.**

Whatever method of construction is selected, the use of a bituminous material by no means justifies any lack of care in the ordinary details to be followed, but rather increases the need for thoroughness and skilled supervision.

The main principles underlying perfection in water-bound roads remain in full force when such roads are treated with bituminous material.

In all uses of bituminous materials in highway work, the use of proper machinery is preferable, wherever conditions make such practicable in any part of the process, to the use of hand labor.

The use or omission of a seal coat on bituminous pavements is dependent on one or more of the following points:

(*a*) The character of the bituminous material used,

(*b*) The method of construction,

(*c*) The density and quality of the work done, and

(*d*) The kind and amount of traffic to be expected on the completed highway.

The length of time that the finished pavement should be closed to traffic varies from a few hours to several days, dependent on the character of the bituminous material used, the method, and on climatic and other local conditions.

Construction—Mixing Methods.—Your Committee is agreed that:

Where the character of the traffic justifies the use of a bituminous concrete pavement, it also demands a correspondingly strong foundation.

The quantity of bituminous material to be used in any case will depend on the peculiar conditions of that case, such as the kind of road metal and of bituminous material, the character of the aggregate, the climatic conditions, etc.

The fluidity of the bituminous materials used should be sufficient to insure a proper coating of the mineral particles, and such fluidity may be obtained by heating—provided the degree of heat is not excessive or sufficient to injure the bituminous materials, and is reached through proper methods.

The bituminous material should possess adhesive and cohesive qualities sufficient to enable it to perform its purposes properly under the conditions of any case, and it should not be of a greasy character.

The adhesiveness, cohesiveness, resiliency, and elasticity of the material should have the greatest possible permanence.

The success of results depends largely on the use of proper sizes of the particles and on securing a proper density of the mixture for the local conditions.

The character of the mineral aggregate to be used may be controlled by local conditions, but the best results can only be obtained by the use of the best materials. Excessive sizes, or excessive variations in the size of the mineral particles, should be avoided, and the utmost care must be taken to avoid the segregation of particles of different sizes.

Mixing machines should be used, and hand-mixing methods should be avoided wherever practicable.

The success of results is greatly dependent on the degree of uniformity obtained in spreading the mixture and on the proper rolling and compacting of the mixture. Too heavy a roller should not be used.

Care should be taken in prescribing or laying any seal coat that invitation or excuse is not given for inferior prior work and its concealment by such a seal coat.

The use of fine sand on top of bituminous concrete is open to grave objections, and the use of clean stone chips or small gravel free from particles that will pass through a 10-mesh sieve seems preferable.

Trap rock in sizes greater than that passing a 2-in. screen should be used with caution in the construction of the upper course unless voids of the same are properly reduced, because of the likelihood of the individual stones to rock under traffic.

In the use of a heated aggregate for the construction of a bituminous concrete pavement non-uniformity or excess in the heating of stone should be avoided.

Construction—Penetration Method.—Your Committee is agreed that:

An important factor for successful results is the proper compaction by rolling of the road metal before the spreading of the bituminous material.

The use of pressure distributors is to be preferred to hand-pouring applications.

The bituminous material should be sufficiently, but not excessively, fluid when used.

The bituminous material should be sufficiently adhesive and cohesive to carry out the purpose for which it is used, and should not be of a greasy character.

The essential characteristics of a bituminous material, such as adhesiveness, cohesiveness, resiliency, and elasticity, should have the greatest possible permanence.

The needed penetration of the material into the rolled stone is had when the bituminous material reaches to the bottom of the average sized stone in the course penetrated.

With the use of materials susceptible to heat, or fairly so, or otherwise of considerable fluidity during application or for a reasonable period thereafter at normal temperatures, the application of the bituminous material to the rolled stone can well be stopped on a line parallel to and within the edges of the stone, and reliance may be had for this border of the macadam becoming properly bound by the flowing of the bituminous material into it under heat and traffic. With the use of materials having the opposite characteristics, the application of the bituminous materials should be made fully to the edges of the rolled stone.

Present indications are to the effect that the use of bituminous materials in quantities of more than 2 1/2 gal. per sq. yd., where the upper course of the macadam is to be 3 in. in thickness after compaction, is inadvisable under the penetration method, although sufficient data do not yet seem to be at hand to determine exactly in any case the value or possible advantage of using bituminous material in excess during construction in order to furnish a sort of reservoir by means of which to prolong the life of the surface of the bituminous pavement.

The use of fine sand on top of the bituminous material is open to grave objections, and clean stone chips or small gravel, free from particles that will pass through a 10-mesh sieve, is preferable.

Construction—Surface Treatments.—Your Committee is agreed that:

The success of surface applications is proportionate to the adhesion obtained by the bituminous material to the old roadway surface, and such degree of cleanliness of the latter must be had,

by means of sweeping or other methods, as will insure a proper degree of adhesion.

The roadway surface should be dry when the bituminous material is applied.

Whether or not the application of bituminous material is to be followed by an application of mineral material, and the length of time to be allowed to elapse between the application of bituminous material and a subsequent application of mineral material, depends on local conditions and on the character and amount of the bituminous material used.

To be successful, a carpet must be able to absorb shocks sufficiently to prevent disintegration, by such shocks, of the surface to which it is applied.

The bituminous material should be sufficiently, but not excessively, fluid when used.

The bituminous material should be sufficiently adhesive and cohesive to carry out the purpose for which it is used under any conditions, and should not be of a greasy character.

The adhesiveness, cohesiveness, resiliency, and elasticity of a bituminous material used for this purpose should be of the greatest possible permanence.

The application of bituminous materials should in all cases be carried well over the outside edges of the rolled metal and on the shoulder far enough to protect the edges of the metaled surface.

The use of fine sand on top of the bituminous material is open to grave objections, the use of clean stone chips or small gravel, free from particles that will pass through a 10-mesh sieve, being generally preferable.

Attempts to construct and maintain by bituminous surface treatments a carpet of a greater total thickness than ½ in. have generally been unsuccessful, and in the construction of a successful carpet of a total thickness approximating ½ in. the materials should be applied in layers, at intervals, rather than by the construction of the total thickness in one application.

The quantity of bituminous material to be used depends on the local conditions in any particular case, but excess in any application is to be avoided, as a deficiency may be corrected by a subsequent application, whereas an excess in any application is difficult, if not impossible, of correction.

The success of results depends first on the condition of the surface to which the treatment is applied. Even the best surface treatment cannot correct defects in the original surface, though it may reduce or modify them.

The application of the bituminous material by means of a pressure distributor is, wherever practicable, preferable to gravity or hand-pouring methods.

Considerable evidence seems to make undesirable the presence of free carbon beyond a maximum, dependent on the distillate between 170° and 300° cent., in tars for superficial treatments, in order at least to avoid brittleness at low temperatures.

The maximum quantity of residue allowable in an asphaltic oil used for superficial treatment should be as great as permitted by the physical conditions of its use, subject to certain limits in the penetration of this residue, and such limits in penetration should be

A minimum which will ensure against undesirable hardness or slipperiness under travel at low temperatures,

A maximum which will ensure against undesirable softness or lack of body at the highest temperatures to be expected after use on the road.

Maintenance and Repairs.—Your Committee is agreed that:

Repairs to bituminous pavements and bituminous surfaces should be made so as to secure immediately the greatest possible homogeneity.

It is even more necessary than with many other types of construction to carry out repairs on bituminous pavements and bituminous surfaces immediately the desirability for them appears.

Nomenclature.—Your Committee wishes to advise strongly the discontinuance of the use of the terms "liquid asphalt" and "asphaltic contents", as these terms are meaningless in many cases, and unnecessary when proper tests, such as are suggested by your Committee, are used.

For the sake of uniformity and clearness, the Committee recommends that the following lists of terms of frequent use in expressions relating to highway work be recognized as having the meanings set forth in the list, unless otherwise definitely stated at any time by a user of such term or terms. The Committee further recommends, that new terms or phrases, coined with the idea of expressing delicate shades of meaning through the medium of one or two words rather than by longer expressions, be suggested or used only after full, careful, and conservative consideration for the sake of avoiding to the utmost further confusion in the use of terms.

In the list given will be found some terms and definitions adopted by the American Society for Testing Materials noted thus *, others by the American Reporters on Communication No. 10 at the Third International Road Congress, designated thus †, and other terms and definitions which have been proposed by the Committee on "Standard Tests for Road Materials" (Committee D 4) of the American Society for Testing Materials, which have been indicated thus ‡. The Committee wishes to acknowledge here its obligations for suggestions thus reaching it.

DEFINITIONS.

***Aggregate.*—The mineral material, such as sand, gravel, shells, slag, or broken stone, or combinations thereof, with which the cement or the bituminous material is mixed to form a mortar or concrete. Fine aggregate may be considered as the mineral inert material which will pass a 1/4-in. screen, and coarse aggregate the material which will not pass a 1/4-in. screen.**

Asphalt.—† Solid or semi-solid native bitumens, solid or semi-solid bitumens obtained by refining petroleums, or solid or semi-solid bitumens which are combinations of the bitumens mentioned with petroleums or derivatives thereof, which melt on the application of heat, and which consist of a mixture of hydrocarbons and their derivatives of complex structure, largely cyclic and bridge compounds.

***Asphalt Block Pavement.*—One having a wearing course of previously prepared blocks of asphaltic concrete.**

***Asphalt Cement.*—A fluxed or unfluxed asphaltic material, especially prepared as to quality and consistency, suitable for direct use in the manufacture of asphaltic pavements, and having a penetration of between 5 and 250.**

Asphaltenes.—† The components of the bitumen in petroleum, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulphide, but insoluble in paraffin naphthas.

***Asphaltic.*—Similar to, or essentially composed of, asphalt.**

Base.—Artificial foundation.

Binder.—(1) A foreign or fine material introduced into the mineral portion of the wearing surface for the purpose of assisting the road metal to retain its integrity under stress, as well as, perhaps, to aid in its first construction. (2) The course, in a sheet-asphalt pavement, frequently used between the concrete foundation and the sheet-asphalt mixture of graded sand and asphalt cement.

Bitumen.—* A mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, which may be gases, liquids, viscous liquids, or solids, and which are soluble in carbon disulphide.

***Bituminous Cement.*—A bituminous material suitable for use as a binder having cementing qualities which are dependent mainly on its bituminous character.**

Bituminous Concrete Pavement.—One composed of stone, gravel, sand, shell, or slag, or combinations thereof, and bituminous materials incorporated together by mixing methods.

***Bituminous Macadam Pavement.*—One having a wearing course of macadam with the interstices filled by penetration methods with a bituminous binder.**

***Bituminous Material.*—Material containing bitumen as an essential constituent.**

***Liquid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 50 grammes applied for 1 sec. of more than 350.**

***Semi-Solid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 100 grammes applied for 5 sec. of more than 10, and under a load of 50 grammes applied for 1 sec. of not more than 350.**

***Solid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 100 grammes applied for 5 sec. of not more than 10.**

Bituminous Pavement.—One composed of stone, gravel, sand, shell or slag, or combinations thereof, and bituminous materials incorporated together.

Bituminous Surface.—A superficial coat of bituminous material with or without the addition of stone or slag chips, gravel, sand, or material of similar character.

Blanket.—See "Carpet".

Bleeding.—The exudation of bituminous material on the roadway surface after construction.

Blown Petroleums.—† Semi-solid or solid products produced primarily by the action of air upon originally fluid native bitumens which are heated during the blowing process.

***Bond.*—The combined action of inertia, friction, and of the forces of adhesion and cohesion which helps the separate particles composing a crust or pavement to resist separation under stress. Mechanical bond is the bond produced almost wholly, in a well-built broken-stone macadam road, by the interlocking of angular fragments of stone and the subsequent filling of the remaining interstices with the finer particles.**

Bound.—Bonded.

Water-Bound.—Bonded with the aid of water.

Bituminous Bound.—Bonded with the aid of bituminous material.

***Brick Pavement.*—One having a wearing course of paving bricks or blocks.**

Bridge.—A structure for the purpose of carrying traffic over a gap in the road-bed measuring 10 ft. or more in the clear span.

Camber of a Road.—See "Crown".

Camber of a Bridge.—The rise of its center above a straight line through its ends.

Carbenes.—† The components of the bitumen in petroleums, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulphide, but insoluble in carbon tetrachloride.

Carpet.—**A bituminous surface of appreciable thickness, generally formed on top of a roadway by the application of one or more coats of bituminous material with gravel, sand, or stone chips added.**

Cement.—**An adhesive substance used for uniting particles of other materials to each other. Ordinarily applied only to calcined "cement rock", or to artificially prepared, calcined, and ground mixtures of limestone and silicious materials. Sometimes used to designate bituminous binder used in bituminous pavements, when the expression "bituminous cement" (q. v.) is understood to be meant.**

Cement-Concrete.—**An intimate mixture of gravel, shell, slag, or broken stone particles with certain proportions of sand or similar material, cement, and water, made previous to placing.**

Cement-Concrete Pavement.—**One having a wearing course of hydraulic cement concrete.**

Cemented.—**Bonded. Referring to water-bound macadam, the term "cemented" is used to designate that condition existing when, after rolling the stone forming the crust, the remaining voids have been filled with the finer sizes, and the stone dust or "flour" has, under the action of water, taken a "set", as does cement itself.**

Chips.—**Small angular fragments of stone containing no dust.**

Clay.—**Finely divided earth, generally silicious and aluminous, which will pass a 200-mesh sieve. Also see "Gravel".**

Coal-Tar.—† The mixture of hydrocarbon distillates, mostly unsaturated ring compounds, produced in the destructive distillation of coal.

Coat.—**See "Carpet". (1) The total result of one or more single surface applications. (2) To apply a coat.**

Coke-Oven Tar.—† Coal-tar produced in by-product coke ovens in the manufacture of coke from bituminous coal.

Consistency.—‡ **The degree of solidity or fluidity of bituminous materials.**

Course.—**One or more layers of road metal spread and compacted separately for the formation of the road or pavement. Courses are usually referred to in the order of their laying as first course, second course, third course, etc. Also a single row of blocks in a pavement.**

Crown.—**The rise in cross-section from the lowest to the highest part of the finished roadway. It may be expressed either as so many inches (or tenths of a foot), or as a rate per foot of distance from side to center, *i. e.*, "the crown is 4 in.", or "the crown is 1/2 in. to the foot".**

Crusher Run.—The total unscreened product of a stone crusher.

Crusher-Run Stone.—**The product of a stone-crusher, unscreened except for the removal of the particles smaller than remaining on about a 1/4-in. screen.**

***Crust.*—That portion of a macadam or similar roadway above the foundation consisting of the road metal proper with its bonding agent or binder.**

Culvert.—A structure for the purpose of carrying traffic over a gap in the road-bed, measuring less than 10 ft. in clear span.

***Cut-Back Products.*—Petroleum, or tar residuums, which have been fluxed, each with its own or similar distillates.**

Dead Oils.—* Oils, with a density greater than water, which are distilled from tars.

Dehydrated Tars.—† Tars from which all water has been removed.

***Ditch.*—The open-side drain of a roadway, usually deep in proportion to its width, and unpaved.**

Drainage.—Provision for the disposition of water.

***Side-Drainage.*—That along the sides of the roadway.**

Sub- or Under-Drainage.—That below the surface.

***Surface Drainage.*—That on the roadway or ground surface.**

***V-Drainage.*—That provided by the construction of troughs in the sub-grade of the roadway, which troughs are like a "V", with flat sloping sides, and are filled with stone.**

***Dust.*—‡ Earth or other matter in fine, dry particles, so attenuated that they can be raised and carried by air currents. The product of the crusher passing through a fine sieve.**

***Dust Layer.*—Material applied to a roadway for temporarily preventing the formation or dispersion under traffic of distributable dust.**

***Earth Road.*—A roadway composed of natural earthy material.**

***Emulsion.*—A combination of water and oily material made miscible with water through the action of a saponifying or other agent.**

***Expansion Joint.*—A separation of the mass of a structure, usually in the form of a joint filled with elastic material, which will provide opportunity for slight movement in the structure.**

***Fat.*—Containing an excess. A fat asphalt mixture is one in which the asphalt cement is in excess and the excess is clearly apparent.**

***Filler.*—(1) Relatively fine material used to fill the voids in the aggregate. (2) Material used to fill the joints in a brick or block pavement.**

***Fixed Carbon.*—* The organic matter of the residual coke obtained upon burning hydrocarbon products in a covered vessel in the absence of free oxygen.**

***Flour.*—† Finely ground rocks or minerals pulverized to an impalpable product.**

Flush Coat.—See "Seal Coat".

Flushing.—(1) Completely filling the voids. (2) Washing a pavement with an excess of water.

Flux.—† Bitumens, generally liquid, used in combination with harder bitumens for the purpose of softening the latter.

Footway.—The portion of the highway devoted especially to pedestrians. A sidewalk.

Foundation.—The portion of the roadway below and supporting the crust or pavement.

Artificial Foundation.—That layer of the foundation especially placed on the sub-grade for the purpose of reinforcing the supporting power of the latter itself, and composed of material different from that of the sub-grade proper.

Free Carbon.—* In tars, organic matter which is insoluble in carbon disulphide.

Gas-House Coal-Tar.—† Coal-tar produced in gas-house retorts in the manufacture of illuminating gas from bituminous coal.

Grade.—(1) The profile of the center of the roadway, or its rate of rise or fall. (2) Elevation. (3) To establish a profile by cuts and fills or earthwork. (4) To arrange by sizes, broken stone, gravel, sand, or combinations of such materials.

Gravel.—Small stones or pebbles usually found in natural deposits more or less intermixed with sand, clay, etc., but in which mixture, the particles which will not pass a 10-mesh sieve predominate, gravel clay, gravel sand, clayey gravel, and sandy gravel indicate the varying proportions of the finer-sized particles. The differentiation between gravel, sand, silt, and clay should be made on the following basis:

Size of Particles.	*Names.*
Retained on a 10-mesh sieve	Gravel.
Passing a 10-mesh and held on a 200-mesh sieve	Sand.
Passing a 200-mesh sieve	Silt or Clay.

Pea Gravel.—Clean gravel the particles of which equal peas in size.

Grit.—Stone, slag chips, or small gravel free from finer material.

Gutter.—The artificially surfaced and generally shallow waterway provided usually at the sides of the roadway for carrying surface drainage. Occasionally used synonymously with "ditch", but incorrectly so, as "gutters" are always paved or otherwise surfaced, and ditches are not.

Haunches.—The sides or flanks of a roadway. Sometimes also called "quarters".

Highway.—The entire right of way devoted to public travel, including the sidewalks and other public spaces, if such exist.

Layer.—A course made in one application.

Loam.—Finely divided earthy material containing a considerable proportion of organic matter.

Macadam.—A road crust composed of stone or similar material broken into irregular angular fragments compacted together so as

to be interlocked and mechanically bound to the utmost possible extent.

***Mastic.*—A mixture of bituminous material and fine mineral matter suitably made for use in highway construction and for application in a heated condition.**

Mat.—† See "Carpet".

***Matrix.*—A composition or material forming a cushion, or binding the aggregate together.**

***Mesh.*—The square opening of a sieve.**

Metal.—See "Road-Metal".

***Mortar.*—A mixture of fine material such as sand, cement, and water or other liquid suitably proportioned and incorporated together for the purpose for which it is used.**

***Mush.*—A greasy mud sometimes found on bituminous crusts.**

***Normal Temperature.*—‡ In laboratory investigations, 25° cent. (77° Fahr.).**

Oil-Gas Tars.—Tars produced by cracking oil vapors in the manufacture of oil gas.

Patching.—Repairing or restoring small isolated areas in the surface of the metaled or paved portion of the highway.

***Palliative.*—A short-lived dust layer.**

***Pavement.*—The wearing course of the roadway or footway, when constructed with a cement or bituminous binder, or composed of blocks or slabs, together with any cushion or "binder" course.**

***Penetration.*—In laboratory investigations, the distance, expressed in tenths of a millimeter, entered a sample by a No. 2 cambric needle operated in a machine for the purpose and under known conditions of loading, time, and temperature. The degree of solidity of bituminous materials.**

In construction, the entrance of bituminous material into the interstices of the metal of the roadway.

Penetration Method.—The method of constructing a bituminous-macadam pavement by pouring or grouting the bituminous material into the upper course of the road metal before the binding of the latter has been completed.

***Pitch.*—† Solid residue produced in the evaporation or distillation of bitumens, the term being usually applied to residue obtained from tar.**

***Hard Pitch.*—Pitch showing a penetration of not more than ten.**

***Soft Pitch.*—Pitch showing a penetration of more than ten.**

***Straight-Run Pitch.*—‡ A pitch run in the initial process of distillation, to the consistency desired without subsequent fluxing.**

***Pocket.*—A hole or depression in the wearing course.**

***Pot-Hole.*—A hole extending below the wearing course.**

***Profile.*—A longitudinal section of a highway, generally taken along the center line.**

***Quarters.*—The four sections of equal width which, side by side, make up the total width of a roadway.**

***Raveling.*—The loosening of the metal composing the crust.**

Refined Tar.—† A tar freed from water by evaporation or distillation which is continued until the residue is of desired consistency, or a product produced by fluxing tar residuum with tar distillate.

Renewals.—Extensive repairs over practically the whole surface of the metaled or paved portion of the highway.

Repairs.—The restoration or mending of a considerable amount of the metaled or paved portion of the highway, but not usually of a majority of the surface area. More extensive than "Patching" but less so than "Renewals".

***Resurfacing.*—The renewal of the surface of the crust or pavement.**

***Road.*—A highway outside of an urban district.**

***Road-Bed.*—The natural foundation of a roadway.**

Road Metal.—Broken stone, gravel, slag, or similar material used in road and pavement construction and maintenance.

***Roadway.*—That portion of a highway particularly devoted to the use of vehicles.**

***Rock Asphalt.*—Sandstone or limestone naturally impregnated with asphalt.**

***Rock Asphalt Pavement.*—A wearing course composed of broken or pulverized rock asphalt with or without the addition of other bituminous materials.**

***Sand.*—Finely divided rock detritus the particles of which will pass a 10-mesh and be retained on a 200-mesh screen. Also see "Gravel".**

***Sand-Clay Road.*—A roadway composed of an intimate mixture of sand and clay.**

***Scarify.*—To loosen and disturb superficially.**

***Screen.*—In laboratory work an apparatus, in which the apertures are circular, for separating sizes of material.**

***Screenings.*—Broken rock of a size that will pass through a 1/2- to 3/4-in. screen, depending on the character of the stone.**

Seal Coat.—A final superficial application of bituminous material during construction to a bituminous pavement.

***Setting Up.*—The relatively quick change such as takes place in a bituminous material after its application to a roadway, indicated by its hardening after cooling and exposure to atmospheric and traffic conditions, as opposed to the slower changes later occurring gradually and almost imperceptibly.**

***Shaping.*—Trimming up and preparing a sub-grade preparatory to applying the first course of the road metal or artificial foundation.**

***Sheet-Asphalt Pavement.*—One having a wearing course composed of asphalt cement and sand of predetermined grading, with or without the addition of fine material, incorporated together by mixing methods.**

***Sheet Pavement.*—A pavement free from frequent joints such as would accompany small slabs or blocks, and which has an appreciable thickness (say, in excess of 1 in. on the average) for its wearing course.**

Shoulders.—The portion of the highway between the edges of the road metal or pavement and the gutters, slopes, or watercourses.

Side Drain.—See "Drainage".

Sidewalk.—The portion of the highway reserved for pedestrians.

***Sieve.*—In laboratory work an apparatus, in which the apertures are rectangular, for separating sizes of material.**

***Silt.*—Naturally deposited fine earthy material, which will pass a 200-mesh sieve. Also see "Gravel".**

***Spalls.*—Fragments broken off by a blow, irregular in shape, and of sufficient size to be comparable to the original mass.**

***Squegee.*—A tool with a rubber or leather edge for scraping or cleaning hard surfaces, or for spreading and distributing liquid material over and into the superficial interstices of roadways.**

***Squegee Coat.*—An application by means of the squegee.**

***Stone Block Pavement.*—One having a wearing course composed of stone blocks quite or nearly rectangular in shape.**

***Street.*—A highway in an urban district.**

Sub-Grade.—The upper surface of the native foundation on which is placed the road metal or the artificial foundation, in case the latter is provided.

Superficial Coat.—A light surface coat.

Surface Coat.—See "Carpet".

***Surfacing.*—(1) The crust or pavement. (2) Constructing a crust or pavement. (3) Finally finishing the surface of a roadway. (4) Treating the surface of a finished roadway with a bituminous material.**

***Surface Treatment.*—Treating the finished surface of a roadway with bituminous material.**

Tailings.—Stones which after going through the crusher do not pass through the largest openings of the screens.

***Tar.*—† Bitumen which yields pitch upon fractional distillation and which is produced as a distillate by the destructive distillation of bitumens, pyro-bitumens, or organic material.**

Telford.—Properly an artificial foundation advocated by Thomas Telford (1757-1820), and consisting of a pavement of stone about 8 in. thick, laid by hand, and closely packed and wedged together. The individual stones were desired to be about 16 sq. in. in section, and about 8 in. in length. They were set close together on the prepared

sub-grade, their longest dimension vertical and on their larger ends, their interstices chinked with smaller stones, and the whole rammed (or rolled) until firm and unyielding.

Telford Macadam.—Macadam with an artificial foundation of Telford.

Under-Drain.—See "Drainage".

***Up-Keep.*—Maintenance.**

V-Drain.—See "Drainage".

***Viscosity.*—The degree of fluidity of bituminous materials.**

***Volatile.*—Applied to those fractions of bituminous materials which will evaporate at climatic temperatures.**

***Water-Bound.*—Bound or bonded with the aid of water.**

Water-Gas Tars.—Tars produced by cracking oil vapors in the manufacture of carbureted water-gas.

***Wearing Coat.*—The superficial layer of the crust or pavement exposed to traffic.**

***Wearing Course.*—The course of the crust or pavement exposed to traffic.**

***Wood Block Pavement.*—One having a wearing course composed of wood paving blocks, generally rectangular in shape.**

Your Committee has carefully considered the advisability of presenting a progress report on "Materials Other than Bituminous". Owing to the time which it has been obliged to devote to the above report on "Bituminous Materials" it has been unable to consider any but the most general questions relating to other materials. Your Committee has been impressed with the fact that there is a wide divergence of opinion and of practice with respect to the use of such materials, but that it might attempt to present some standards and principles which are fairly well defined and quite generally accepted, and to suggest other matters which should receive attention and be made the subject of investigation by highway engineers. The majority of the Committee feels, however, that it should defer a progress report on "Materials Other than Bituminous" until it is able to secure more precise information and reach more definite conclusions.

The Committee wishes to express again its deep appreciation of the assistance rendered it by the Board of Direction and by your Secretary, Mr. Charles Warren Hunt, as well as by the members of the Society and others.

Very respectfully,

SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION, ETC.

W. W. CROSBY, *Chairman,*
H. K. BISHOP,
A. W. DEAN,
N. P. LEWIS,
C. J. TILDEN,
G. W. TILLSON,
A. H. BLANCHARD, *Secretary.*

APPENDIX A

Forms Originally Prepared and Adopted by the Special Committee, American Society of Civil Engineers, on "Bituminous Materials for Road Construction, and on Standards for Their Test and Use", in 1909.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Special Committee on

Bituminous Materials for Road Construction.

July 31st, 1909.

Dear Sir:

The importance of a better knowledge of the various properties of such Bituminous Materials as are now on the market and of those likely to be used in Road Construction in the near future, and of a clearer understanding of the effects of these properties when such materials have been so used, has doubtless impressed you before this. With a view toward obtaining for the general benefit, conclusions of value at the earliest possible moment, the undersigned have been appointed a Special Committee to investigate and report on Bituminous Materials for Road Construction and on Standards for Their Test and Use.

The Committee has, after due deliberation, concluded that the information called for on the attached form is most desirable for a proper and complete consideration of the matter in hand and in order that clear and definite conclusions may be reached and reported by the Committee. To this end, therefore, the Committee would respectfully request that you furnish on the attached form all the information you find it practicable to express. The Committee realizes fully that the methods of recording information concerning the work in different places may vary considerably and that it may not be possible in all or even many cases to fill in completely the blanks of the form arranged by the Committee. However, the attempt has been made to arrange the form so as to provide for summations where subdivided information shall be impracticable of expression, and the Committee begs to assure you that, even if information in all details cannot be furnished, partial information and figures, such as it shall seem practicable for you to furnish, will be greatly appreciated. It is the opinion of the Committee that the collection of even partial information from various sources may be of great assistance in making clearer many matters now in dispute.

In order for the work of the Committee to be of value to the greatest extent, it is, of course, advisable that it carefully consider in detail

as much information from many sources as it is possible to secure, and the Committee therefore hopes that it may be favored with your co-operation by the furnishing of the information requested. Detailed explanations regarding the filling of the blanks in the form will be found on the last page of the same.

The Committee begs to express here its appreciation of such information and co-operation as you may be able to give it and to subscribe itself,

Very respectfully,

W. W. CROSBY, *Chairman,*
H. K. BISHOP,
A. W. DEAN,
A. H. BLANCHARD, *Secretary,*
Committee.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

SPECIAL COMMITTEE ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.

DATA CONCERNING THE USE OF BITUMINOUS MATERIALS IN ROAD SURFACING.

GENERAL INFORMATION.

State .....County .....
Town .....Road .....
Contractor .....
Length of road.....
Width of metal surfacing (average).....
Area of " " (total) .....
Thickness of surfacing after rolling, with area of each:
2".....sq. yds.; 4".....sq. yds.; 6".....sq. yds.
8"....." " ; 10"....." " ; 12"....." " "
Amount of crown, maximum.....minimum.....
Nature of subgrade (1).....
Percentage of grade (maximum) Length.....ft.....per cent.
" " " (minimum) " ....." ....." " "
" " " (mean) (2) " ....." ....." " "
Date of beginning of construction of metal surfacing.....
" " completion of " " " " .....
Average rate of progress of construction of metal surfacing per working day .....sq. yds.
Average number of hours per working day.....hours.

CLASS OF HIGHWAY OR NATURE OF TRAFFIC.

.....
.....

AMOUNT OF TRAFFIC OR TRAFFIC CENSUS FOR .... HOURS (AVERAGE) (3).

Dates of census.....

	Empty vehicles.	Loaded vehicles.	Est. (in lbs.) of maximum load per inch of tire.	Passenger vehicles.
Horse-Drawn Vehicle Traffic.				
One-Horse vehicles.....				
Two " "				
Three " "				
Four " "				
Five " "				
Six or more horse vehicles.....				
Motor-Vehicle Traffic.				
Motor cycles				
" runabouts.....				
" touring cars (four or five seats)....				
" touring cars (six or seven seats) including limousines or landaulets.				
" wagons or drays.....				

FORM OF CONSTRUCTION.

(*i. e.*, macadam, gravel, sand or other surface, and whether treated with a bituminous material by mixing, penetration or other method.)

.....

.....

Artificial foundation; nature of (4).....

.....; depth.....

HAULING DISTANCES.

Average length of haul for gravel.....miles.

" " " " " stone ..... "

" " " " " sand ..... "

" " " " " shells ..... "

" " " " " bituminous materials..... "

" " " " " binder or grit..... "

" " " " " .....

" " " " " .....

RATES OF LABOR (per day).

Foreman..... $.....; Roller and engineer... $.....

Labor (ordinary)..... .....; Water cart or sprinkler. .....

" (special)..... .....; Scarifier ....

One-horse team and driver. .....; ....

Two " " " " . .....; ....

Three " " " " . .....; ....

Equipment (5).

Rollers:— number.....kind.....weight.....tons
Water carts:— " ..... " .....capacity .....gallons
Oilers:— " ..... " ..... " ..... "
Scarifiers:— " ..... " .....
.....
.....
Mixing plant.....
.....
.....
Kettles and accessories.....
.....
.....
Distributors .....
.....

Materials.

Bituminous Material.

Kind of material.....
Trade name .....
Purchased from.....
Shipped from.....
Furnished by party of the first or second part.....

Characteristics and Analyses (6).

	Tar compounds.	Asphaltic compounds.
Specific gravity.....		
Water soluble material (organic).....	per cent.	per cent.
" " " (inorganic).....	"	"
Free carbon.....	"	"
Ash.....	"	"
Solubility in cold carbon tetrachloride.....	.....	"
Fixed carbon	per cent.	"
Paraffin	.....	"
Melting point of normal material.....	degrees	degrees
Evaporation 5 hours at 170° C.....	per cent.	per cent.
Melting point of residue.....	degrees	degrees
Penetration of residue at 4° C.....		
" " " " 25° C.....		

CHARACTERISTICS AND ANALYSES (6)—*Continued.*

	Tar compounds.	Asphaltic compounds.
Evaporation 5 hours at 205° C.		per cent.
Melting point of residue		degrees
Penetration of residue at 4° C		
" " " " 25° C		
Solubility in 88° B. naphtha		per cent.
Character of solution (oily or sticky)		
Distillation		
Up to 105° C.	per cent.	
105° C. to 170° C.	"	
170° C. to 225° C.	"	
225° C. to 270° C.	"	
270° C. to 300° C.	"	
Viscosity 100° C. (7)		
Engler viscosimeter	seconds	seconds
Lunge tar tester	"	"
Penetrometer		
New York Testing Laboratory viscosimeter		
Viscosity 25° C. (7)		
Engler viscosimeter	seconds	seconds
Lunge tar tester	"	"
Penetrometer		
New York Testing Laboratory viscosimeter		

CONSTRUCTION DATA.

Amount of bituminous material used per sq. yd. (in gals.).
First course..... second course.....
Surface coat ..... total .....
Temperature to which material was heated.
First course.....second course.....surface coat.....
Temperature of air during use of bituminous material.
Maximum .....minimum .....mean .....
Date of beginning of use of bituminous material.....
Date of completion of " " " " .....
Average rate of progress of using bituminous material (per day in gallons).
First course.....second course.....surface coat.....
General weather conditions during use of material.....
Cost, f. o. b. cars, of bituminous materials (per gallon).....
Freight on bituminous materials (per gallon).....

Cost of loading and hauling bituminous materials (per gallon).....
Total cost delivered on road of bituminous materials (per gallon).....
Cost of heating " " " " .....
Cost of spreading or mixing " " " " .....
First course.....second course.....surface coat.....
Total cost in place of bituminous materials (per gallon).....
" " " " " " " (" sq. yd.).....
Contract price in place of bitufminous materials (per sq. yd.).....

METAL SURFACING.

	Telford or other base.	1st course.	2nd course.	Binder or grit.	Totals.
Name (8) of material used.....					
Abrasion test (9) of material used.....					
Absorption test of material used.....					
Fracture (10) of material used.....					
Crushing strength of material used (lbs. per sq. in.).....					
Thickness after rolling of material used.					
Size (11) of material used.....					
Amount (in tons) or cu. yds. (12) of material used.....					
Cost, f. o. b. cars, of material used, per ton or cu. yd. (12).....					
Freight on material used, per ton or cu. yd. (12).....					
Cost of loading and hauling material used, per ton or cu. yd. (12).....					
Total cost delivered on road of material used, per ton or cu. yd. (12).....					
Total cost delivered on road of material used, per sq. yd.....					
Cost of scarifying and shaping old surface, per sq. yd.....					
Cost of laying or spreading of material used, per sq. yd.....					
Cost of watering and rolling of material used, per sq. yd.....					
Total cost in place of material used, per sq. yd.....					
Contract price in place of material used, per sq. yd.....					
.....					
.....					

REMARKS.

.....
.....

NOTES AND EXPLANATIONS.

(1) In stating "nature of subgrade" note whether sandy, clayey, loamy, or otherwise and whether readily and thoroughly compacted by rolling.

(2) "Mean" percentage of grades to be obtained by multiplying length of grade by its per cent., adding all products together and dividing by total length.

(3) If possible "average" should be arrived at from two or three separate determinations. State number of determinations from which "average" is secured.

(4) After "Artificial Foundation, nature," sufficient information should be inserted to indicate whether simply a layer of gravel is used, and nature (sandy or otherwise) of same; or whether a foundation is laid by hand of large stone; or whether a layer of crushed stone is rolled into the subgrade before commencement of surfacing proper.

(5) Under "Equipment" should be stated any charges paid as rentals.

(6) For methods of determination of characteristics listed, see enclosed "List of Analyses and Methods of Testing Bituminous Materials" proposed by Committee.

(7) In reporting "Viscosities" of course figures can only be given in terms of the one (or more) instruments applying in any case.

(8) Give, if possible, geological "name" of material used in each course. Reference, "Rocks," by Kemp.

(9) "Abrasion test" should be stated in terms of French coefficient of wear.

(10) "Fracture" should be stated as "cubical" or "scaly."

(11) "Size" should be stated as maximum and minimum of sizes of openings in screens to pass and hold stone used.

(12) Cross out unit not used.

Name .....
Title .....
Address .....
.....
.....
Reporting Officer.

Date ....., 19.....

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Special Committee on Bituminous Materials for Road Construction.

List of Analyses and Methods of Testing Bituminous Materials Proposed by Committee.

July, 1909.

Tars.

Water-Soluble Materials.—Boil gently 2 grams of material with 25 c.c. of distilled water for one hour. Filter and wash with 25 c.c. of boil-

ing water. Evaporate filtrate in weighed dish to dryness and constant weight at 105° C. Weigh residue. Ignite residue and weigh again, giving weight of inorganic matter plus weight of crucible. Weight No. 2 minus weight No. 3 gives weight of organic matter.

Specific Gravity.—Use some standard form of pyknometer. Material and distilled water must have a temperature of 25° C.

For semi-solid and solid materials use Sommer's Pyknometer.

Free Carbon.—The free carbon shall be determined by dissolving for 15 hours, 2 grams of the compound in 100 c.c. of cold carbon bisulphide, filtering the solution through a weighed Gooch crucible, fitted with an asbestos pad, drying to constant weight, and weighing the insoluble residue; then igniting crucible until all carbon is burned off, weighing the residue (ash). The difference between the 2nd and 3rd weights is "Free Carbon." The difference between the 1st and 3rd, is ash, which should be noted.

Fixed Carbon.—About one gram of the compound is weighed into a platinum crucible one and one-eighth to one and one-half inches high. The crucible with the lid on is heated, first gently, and then until no more smoke and flame issues between the crucible and the lid. It is then heated three and one-half minutes in the full heat of the burner; then cooled and weighed. The crucible lid is then removed and the crucible and contents allowed to remain in the full heat of the burner until the carbon is burned off, and then weighed again. The difference between these two weights is the Fixed Carbon.

Evaporation.—20 grams of compound are heated in a flat-bottomed dish, two and one-half inches in diameter and about one inch high, for a total of five hours in three successive periods of three, one and one hours, respectively, in an oven, the interior of which is maintained at a uniform and constant temperature of 170° C. This oven is to be controlled by any thermo regulator, controlling within two degrees, and is to have its full temperature before the compound is introduced. The dish must be level. Remove dish from oven and stir contents thoroughly for one minute between successive periods.

Penetration of Residue from Evaporation Test.—The penetration shall be measured by a standard machine using 100 grams load and No. 2 needle. Use a flat-bottomed glass dish seven-eighths of an inch in diameter and one and one-half inches in height. Fill flush with top with material and allow same to stand at room temperature for one-half hour. Immerse in water bath, covering material for one hour. Immerse needle to be used for five minutes in same bath. Test at once, making three determinations. The recorded penetration will be the average value. Temperature 4° C. and 25° C.

(Note): Residue must be melted at lowest possible temperature and thoroughly mixed by stirring.

Melting Point of Residue from Evaporation.—The material whose melting point is to be determined, is melted and poured into a mold that will make a one-half inch cube. A No. 10 gauge wire about 6″ to 8″ long is bent at right angles for a length of ¾″ at one end and the center of the cube is placed on this end so that one of the diagonals of the vertical face of the cube is parallel to the long part of the wire. Take a bottle of a size about 2″ in diameter and 4″ high and place a piece of white paper in the bottom of it. Pass the long part of the wire through the cork of the bottle so that the lower edge of the cube will be within one inch of the bottom of the bottle. Also put a thermometer through the cork so that the bulb is opposite the cube. Place the bottle in a water or oil bath and raise the temperature of the bath at a rate of three to six degrees C. a minute. The melting point of the material is the temperature of the thermometer inside the bottle at the time that the material touches the paper in the bottom of the bottle.

Distillation.—
Up to 105° C.
From 105° to 170° C.
From 170° to 225° C.
From 225° to 270° C.
From 270° to 300° C.

700 grams of the compound are weighed into a retort (E. & A. four pints, No. 4 521), whose top is fitted with a tee as close as possible to the retort, and a condenser pipe 24″ to 36″ long; the upper branch of the tee is used for the insertion of a thermometer, the top of whose bulb is placed immediately below the main outlet of the tee.

Viscosity or Consistency.—Temperatures at which viscosities will be determined, are 100° C. and 25° C.

Penetrometer to be used in accordance with standard method on materials solid at above temperatures. On materials which at the above temperature, the penetrometer cannot be used, the viscosity shall be determined by one of the following instruments:

Engler Viscosimeter.
Lunge Tar Tester.
New York Testing Laboratory Viscosimeter.

COMPOUNDS PREPARED FROM PETROLEUM OR NATURAL ASPHALT PITCHES.

Melting Point of Solid Asphalts.—Same method as for residue from evaporation of tars.

Water-Soluble Materials.—Same method as for tars.

Specific Gravity.—Same method as for tars.

Free Carbon.—Same method as for tars.

Material Soluble in Cold Carbon-Tetrachloride.—Same method as for Free Carbon, except carbon-tetrachloride is used as a solvent instead of carbon-bisulphide.

Fixed Carbon.—Same method as for tars.

Paraffin.—100 grams or less of the compound is distilled rapidly in a retort to dry coke.

5 grams of the well mixed distillate is treated in a two-ounce flask with 25 c.c. Squibbs absolute ether; after mixing thoroughly, 25 c.c. Squibbs absolute alcohol is added and the flask packed closely in a freezing mixture of finely crushed ice and salt for at least 30 minutes. Filter the precipitate quickly by means of a suction pump, using a No. 575 C. S. & S. 9 c.m. hardened filter paper. Rinse and wash the flask and precipitate (with 1 to 1 Squibbs alcohol and ether mixture cooled to —17° C.) until free from oil (50 c.c. of washing solution is usually sufficient). When sucked dry remove paper, transfer waxy precipitate to small glass dish, evaporate on steam bath and weigh paraffin remaining on dish.

Calculation.—Weight of paraffin divided by weight of distillate taken and multiplied by per cent. of total distillate used from original sample, equals per cent. of paraffin.

Evaporation Test No. 1.—Same method as for tars.

Penetration of Residue from Evaporation Test No. 1.—Same method as for similar residue of tars.

Melting Point of Residue from Evaporation Test No. 1.—Same method as for similar residue of tars.

Solubility in 88° Baumé Naphtha.—2 grams of compound are placed in 4-oz. oil-sample bottle made up to 100 c.c. with 88° B. naphtha, having a boiling point between 40° C. and 55° C., the whole well shaken until compound is broken up. The bottle is then centrifugalized for 10 minutes, 50 c.c. are withdrawn into a weighed flask, the naphtha distilled by a water bath and the residue weighed. Another 10 c.c. of the naphtha solution is run over 3½″ Petri glass and allowed to evaporate for 24 hours at room temperature. Note character of residue, *i. e.*, sticky or oily.

Viscosity or Consistency.—Same as for tars.

Evaporation Test No. 2.—Same method as for tars except oven temperature shall be 205° C.

Penetration of Residue from Evaporation Test No. 2.—Same method as for tars.

Melting Point of Residue from Evaporation Test No. 2.—Same method as for tars.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

SPECIAL COMMITTEE ON
BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.

To.....
.....
.....

The Committee again acknowledges its indebtedness for co-operation in reporting on the construction of roads, in connection with which bituminous materials were used. The information furnished is of great value, but in order to make it more fully available it seems advisable to request the supplementary information on the points covered by the following form.

The general information previously furnished has been filled in by the Secretary.

It is the intention of the Committee to ask annually for reports, each to cover a year of observation, and it is hoped that this form will be returned as soon as data covering a year are available.

Report on Results of the Use of
BITUMINOUS MATERIALS IN ROAD CONSTRUCTION.

GENERAL INFORMATION.

(Required for identification with previous reports.)

State.....County.....
Town.....Road.....
Contractor.....
Kind of bituminous material.....
Trade name.....
Method.....
Dates between which work was done.....to.....

PERIOD COVERED BY THIS REPORT.

From.....to.....

CLASS OF HIGHWAY OR NATURE OF TRAFFIC.

.....
.....

AMOUNT OF TRAFFIC OR TRAFFIC CENSUS FOR.....HOURS (AVERAGE).*

(Dates of census.....)

	Empty vehicles.		Loaded vehicles.		Est. (in lbs. of) maximum load per inch of tire.		Passenger vehicles.	
Horse-Drawn Vehicle Traffic.	Nov. to March.	Apr. to Oct.	Nov. to March.	Apr. to Oct.	Nov. to March.	Apr. to Oct.	Nov. to March.	Apr. to Oct.
One-Horse vehicles.....								
Two " "								
Three " "								
Four " "								
Five " "								
Six or more horse vehicles.								
Motor-Vehicle Traffic.								
Motor cycles.....								
" runabouts.....								
" touring cars (four or five seats).....								
" touring cars (six or seven seats) including limousines or landaulets.								
" wagons or drays....								

* If possible, "average" should be arrived at from two or three separate determinations. State number of determinations from which average is secured.

(1) Date of opening road to traffic, or average length of time after completion before sections of road were opened to traffic.
(2) Has the surface begun to disintegrate? If so, when and in what manner? What was the probable cause?
(3) Has any displacement of the surface been manifest in the shape of ruts, waves, etc.?
(4) Was the surface objectionably soft in hot weather? If so, state air temperature; also state maximum temperature at which surface was satisfactory.
(5) Is the tractive effort increased by the presence of the bituminous material?
(6) Was the surface slippery for horses or motor cars in (1) cold weather, (2) when wet, (3) on frosty mornings?
(7) Are the stones of the top course of the macadam visible? If so, to what extent?
(8) Has the surface worn differently adjacent to curbs and car rails than it has in the traveled ways?
(9) During what period covered by this report was the surface coated with snow or ice?
(10) Maximum depth of frost in this period.
(11) Amount of rainfall for the period covered by this report.
(12) Has the surface been artificially watered? If so, to what extent?
(13) Has surface been cleaned? If so, to what extent?
(14) Is road shaded or exposed?
(15) Cost and character of maintenance during period covered by this report.
(16) Your general opinion on the results secured on the above road and any information not covered properly by the above is desired.
(Then follows a space for answers to the foregoing questions and for date and name, title, and address of the reporting officer.)

APPENDIX B

Revised Forms Prepared and Adopted by the Special Committee, American Society of Civil Engineers, on "Materials for Road Construction, and on Standards for Their Test and Use", in 1914.

Bituminous Materials for Road Construction and Standards for Their Test and Use.

Special Committee of the American Society of Civil Engineers.

Dear Sir:

Without doubt you fully appreciate the value of a comprehensive record of the details of construction and maintenance of bituminous surfaces and bituminous pavements and of the effects of the various chemical and physical properties of bituminous materials when used in connection with different non-bituminous materials under diverse methods of construction and maintenance of roads and pavements. With a view toward obtaining for the general benefit conclusions of value, the Board of Direction of the Society appointed a Special Committee to investigate and report on "Bituminous Materials for Road Construction and on Standards for Their Test and Use."

This Committee, in continuation of its work of collating information, has issued a second form for use in recording data relative to the construction of bituminous surfaces and bituminous pavements. The Committee has concluded that the information called for on the enclosed form is desirable for a proper consideration of the matter in hand and in order that clear and definite conclusions may be reached. To this end, therefore, the Committee would respectfully request that you furnish on the forms information relative to the bituminous surfaces and bituminous pavements built during the preceding and current construction seasons, using one form for each surface or pavement. The Committee assures you that, even if information on all the details cannot be furnished, partial facts and figures, such as it shall seem practicable for you to furnish, will be greatly appreciated.

In order that the work of the Committee may be of value to the greatest extent, it is advisable that it carefully consider in detail as much information from as many sources as it is possible to secure, and the Committee therefore hopes that it may be favored with your co-operation by the receipt from you of the information solicited.

Thanking you for a consideration of the above request, we remain,

Very respectfully,

By order of the Committee,

August, 1914. A. H. Blanchard, *Secretary.*

Bituminous Materials for Road Construction and Standards for Their Test and Use.

Special Committee of the American Society of Civil Engineers.

Data Concerning the Use of Bituminous Materials in Surfaces and Pavements.

General Information.

State.....County .....
Town or City.....Highway.....
Contractor.....
Length of Work .....Width of Metal (Average).....
Date of Beginning.....Date of Completion.....
.....(of use of Bituminous Materials)
Method of Construction (state character of foundation, thickness of courses, kinds and quantities of road metal and of bituminous materials used, methods used, and approximate maximum grades). (Attach copy of specifications if available.)

Average Daily Traffic as Determined by Census.

Taken for.....Hours on Following Dates.....

	Commercial		Passenger or Pleasure	
	Empty	Loaded		
One-horse vehicles.....				
Two or three-horse vehicles.....				
Four or more horse vehicles.....				
.....				
Motor cycles.....				
" runabouts.....				
" touring cars (open or closed)				
" busses.....				
" trucks.....				

Are widths of tires on horse-drawn vehicles regulated by law or ordinance?
If so, please state regulations on separate sheet.
If no traffic census is available, give estimate of traffic.
.....
Total cost of bituminous surface or pavement, $.....
Cost per square yard, $.....Average thickness, in inches.....
Average wages per day, of common labor, $.....Working day of.....hours.

Description of Bituminous Material used.
(State type, trade name, company purchased from, and if supplied by contractor or otherwise.)
(If analysis is available, please attach copy of same.)
REMARKS: (Including notes as to speed, tire widths, and any preponderance of traffic at certain hours.)

Name .....
Title .....
Address .....
.....
.....
Reporting Officer.

Date.....191..

APPENDIX C

ANALYSES AND TESTS OF BITUMINOUS MATERIALS.

The adoption of the following lists of tests, as including all those probably of value in determining and recording the characteristics of the bituminous materials used in this connection, is recommended:

TARS.

Specific gravity at 25° cent. (77° Fahr.).
Flash point.
Solubility in CS_2 (carbon disulphide).
Consistency at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), 46° cent. (115° Fahr.), 98° cent. (208° Fahr.).
Melting point.
Loss on evaporation at 163° cent. (325° Fahr.).
    Consistency of residue at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), 46° cent. (115° Fahr.), 98° cent. (208° Fahr.).
    Melting point of residue.
Distillation.
    Consistency of residue at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), 46° cent. (115° Fahr.), 98° cent. (208° Fahr.).
    Melting point of residue.

ASPHALTIC MATERIALS.

Specific gravity at 25° cent. (77° Fahr.).
Flash point.
Solubility in CS_2 (carbon disulphide).
Solubility of bitumen in CCl_4 (carbon tetrachloride).
Solubility of bitumen in petroleum naphtha.
    Character of residue on glass.
Consistency at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), 46° cent. (115° Fahr.), 98° cent. (208° Fahr.).
Melting point.
Ductility at 4° cent. (39° Fahr.) and 25° cent. (77° Fahr.).
Fixed carbon content.
Paraffin content.
Loss on evaporation at 163° cent. (325° Fahr.).
    Consistency of residue at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), 46° cent. (115° Fahr.).
    Melting point of residue.
    Ductility of residue at 4° cent. (39° Fahr.) and 25° cent. (77° Fahr.).

It is further recommended that the following methods for performing the tests above listed be adopted as standard:

Specific Gravity.

For liquid and semi-solid materials, some standard form of pyknometer shall be used. For solid materials, the suspension method shall be used. Material and distilled water shall have a temperature of 25° cent. (77° Fahr.).

The pyknometer to be used shall consist of a fairly heavy, straight-walled glass tube, 70 mm. (2.75 in.) long and 22 mm. (0.875 in.) in diameter, ground to receive a solid glass stopper with a hole of 1.6-mm. (0.063-in.) bore, in place of the usual capillary opening. The lower part of this stopper shall be made concave in order to allow all air bubbles to escape through the bore. The depth of the cup-shaped depression is 4.8 mm. (0.188 in.) at the center. The stoppered tube shall have a capacity of about 24 cu. cm. (0.811 oz.) and when empty shall weigh about 28 grammes. Its principal advantages are: (1) that any desired quantity of bituminous material may be poured in without touching the sides above the level desired; (2) it is easily cleaned; (3) on account of the 1.6-mm. (0.063-in.) bore, the stopper can be more easily inserted when the tube is filled with a very viscous oil than if it contained a capillary opening. When testing solid or semi-solid materials with the pyknometer, extreme care should be taken in melting, to avoid loss by evaporation, and, in filling the pyknometer, to avoid entrapping air.

When working with semi-solid bituminous materials which are too soft to be broken and handled in fragments, the following method of determining their specific gravity has been used with good results. The clean, dry pyknometer is first weighed empty and this weight is called *a*. It is then filled in the usual manner with freshly distilled water at 25° cent. (77° Fahr.), and the weight is again taken and called *b*. A small quantity of the material is then placed in a spoon and brought to a fluid condition by the gentle application of heat, with care that no loss by evaporation occurs. When sufficiently fluid, enough is poured into the dry pyknometer, which may also be warmed, to fill it about half full, without allowing the material to touch the sides of the tube above the desired level. The tube and contents are then allowed to cool to room temperature, after which the tube is carefully weighed with the stopper. This weight is called *c*. Distilled water, at 25° cent. (77° Fahr.), is then poured in until the pyknometer is full. After this the stopper is inserted and the whole is cooled to 25° cent. (77° Fahr.) by a 30-min. immersion in a beaker of distilled water maintained at this temperature. All surplus moisture is then removed with a soft cloth, and the pyknometer and contents are weighed. This weight is called *d*. From the weights

obtained, the specific gravity of the material may be readily calculated by the following formula:

$$\left.\begin{matrix}\text{Specific gravity } 25^\circ \text{ cent. } (77^\circ \text{ Fahr.})\\ 25^\circ \text{ cent. } (77^\circ \text{ Fahr.})\end{matrix}\right\} = \frac{c - a}{(b - a) - (d - c)}.$$

Both a and b are constants, and need be determined only once. It is necessary, therefore, to make only two weighings for each determination after the first. Results obtained according to this method are accurate to within two units in the third decimal place, whereas the open-tube method commonly used is accurate to the second decimal place only.

The specific gravity of fluid bituminous material may be determined in the ordinary manner with this pyknometer by completely filling it with the material and dividing the weight of the bituminous material thus obtained by that of the same volume of water.

Flash Point.

The flash point shall be determined by the closed-cup test.

Although for ordinary purposes, the open-cup method of determining the flash and burning points of bituminous materials is reasonably accurate, the closed-cup method described below is to be preferred.

The oil tester shall consist of a copper oil cup having a capacity of about 300 cu. cm. (10.1 oz.), and shall be heated in a water or oil bath by a small Bunsen flame. The cup shall be provided with a glass cover, carrying a thermometer, and a hole for inserting the testing flame. The testing flame shall be obtained from a jet of gas passed through the piece of glass tubing, and shall be about 5 mm. (0.197 in.) in length.

The flash test shall be made as follows: The oil cup shall first be removed and the bath filled with water or cottonseed oil. The oil may always be used, and is necessary for bituminous material flashing at a temperature of more than 100° cent. (212° Fahr.). The oil cup shall be replaced and filled with the material to be tested to within 3 mm. (0.118 in.) of the flange joining the cup and the vapor chamber above. The glass cover shall then be placed on the oil cup and the thermometer adjusted so that its bulk shall be just covered by the bituminous material. The Bunsen flame shall be applied in such a manner that the temperature of the material in the cup shall be raised at the rate of about 5° cent. (9° Fahr.) per min. From time to time the testing flame shall be inserted in the opening in the cover to about half way between the surface of the material and the cover. The appearance of a faint bluish flame over the entire surface of the bituminous material will show that the flash point has been reached and the temperature at this point is taken.

Solubility in Carbon Disulphide (CS_2).

This test shall consist in dissolving the bituminous material in carbon disulphide and recovering any insoluble matter by filtering the solution through an asbestos felt. The Gooch crucible used for the determination shall be 4.4 cm. (1.722 in.) wide at the top, tapering to 3.6 cm. (1.417 in.) at the bottom, and shall be 2.5 cm. (0.984 in.) deep.

The asbestos shall be cut with scissors into pieces not exceeding 1 cm. (0.394 in.) in length, after which it shall be shaken up with just sufficient water to pour easily. The crucible shall be filled with the suspended asbestos and allowed to settle for a few moments. A light suction shall then be applied to draw off all the water and leave a firm mat of asbestos in the crucible. More of the suspended material shall be added, and the operation shall be repeated until the felt shall be so dense that it scarcely transmits light when held so that the bottom of the crucible is between the eye and the source of light. The felt shall then be washed several times with water, and drawn firmly against the bottom of the crucible by an increased suction. The crucible shall be removed to a drying oven for a few minutes, after which it shall be ignited at red heat over a Bunsen burner, cooled in a desiccator and weighed.

Two grammes of bituminous material or 10 grammes of an asphalt topping or rock asphalt shall then be placed in an Erlenmeyer flask, which shall have been weighed previously, and the accurate weight of the sample obtained. One hundred cubic centimeters (3.381 oz.) of chemically pure carbon disulphide shall be poured into the flask, in small portions, with continual agitation, until all lumps disappear and nothing adheres to the bottom. The flask shall then be corked and set aside for 15 min. to allow settlement of the insoluble material.

The weighed Gooch crucible containing the felt shall be set up over the dry pressure flask, and the solution of bituminous material in carbon disulphide shall be decanted through the felt without suction by gradually tilting the flask, with care not to stir up any precipitate that may have settled out. At the first sign of any sediment coming out, the decantation shall be stopped and the filter allowed to drain. A small quantity of carbon disulphide shall then be washed down the sides of the flask, after which the precipitate shall be brought upon the felt and the flask scrubbed, if necessary, with a feather or "policeman", to remove all adhering material. The contents of the crucible shall be washed with carbon disulphide until the washings run colorless. Suction shall then be applied until there is practically no odor of carbon disulphide in the crucible, after which the outside of the crucible shall be cleaned with a small quantity of the solvent. The crucible and contents shall be dried in the hot-air oven at 100° cent. (212° Fahr.) for about 20 min., cooled in a

desiccator, and weighed. If any appreciable quantity of insoluble matter adheres to the flask, it shall also be dried and weighed, and any increase over the original weight of the flask shall be added to the weight of insoluble matter in the crucible. The total weight of insoluble material may include both organic and mineral matter. The former, if present, shall be burned off by ignition at a red heat until no incandescent particles remain, thus leaving the mineral matter or ash, which can be weighed on cooling. The difference between the total weight of material insoluble in carbon disulphide and the weight of substance taken equals the total bitumen, and the percentage weights are calculated and reported as total bitumen, and insoluble organic and inorganic matter, on the basis of the weight of material taken for analysis.

This method is quite satisfactory for straight oil and tar products, but, where native asphalts are present, it will be found practically impossible to retain all the finely divided mineral matter on an asbestos felt. It is generally more accurate, therefore, to obtain the result for total mineral matter by direct ignition of a 1-gramme sample in a platinum crucible, or to use the result for ash obtained in the fixed carbon test. The total bitumen is then determined by deducting from 100% the sum of the percentages of total mineral matter and insoluble organic matter. If the presence of a carbonate mineral is suspected, the percentage of mineral matter may be most accurately obtained by treating the ash from the fixed carbon determination with a few drops of ammonium carbonate solution, drying at 100° cent. (212° Fahr.), then heating for a few minutes at a dull red heat, cooling, and weighing again.

When difficulty in filtering is experienced—for instance, when Trinidad asphalt is present in any quantity—a longer period of subsidence than 15 min. is necessary, and the following method, proposed by the Committee on Standard Tests for Road Materials (Committee D-4) of the American Society for Testing Materials, is recommended: (Note: English equivalents in brackets have been added by the Special Committee on Materials for Road Construction).

From 2 to 15 grammes (depending on the richness in bitumen of the substance) is weighed into a 150-cu. cm. (5.072-oz.) Erlenmeyer flask, the tare of which has been previously ascertained, and treated with 100 cu. cm. (3.381 oz.) of carbon disulphide. The flask is then loosely corked and shaken from time to time until practically all large particles of the material have been broken up, when it is set aside and not disturbed for 48 hours. The solution is then decanted off into a similar flask that has been previously weighed, as much of the solvent being poured off as possible without disturbing the residue. The first flask is again treated with fresh carbon disulphide and shaken as before, when it is put away with the second flask and not disturbed for 48 hours.

At the end of this time the contents of the two flasks are carefully decanted off upon a weighed Gooch crucible fitted with an asbestos filter, the contents of the second flask being passed through the filter first. The asbestos filter shall be made of ignited long-fiber amphibole, packed in the bottom of a Gooch crucible to the depth of not more than $\frac{1}{8}$ in. After passing the contents of both flasks through the filter, the two residues are shaken with more fresh carbon disulphide and set aside for 24 hours without disturbing, or until it is seen that a good subsidation has taken place, when the solvent is again decanted off upon the filter. This washing is continued until the filtrate or washings are practically colorless.

The crucible and both flasks are then dried at 125° cent. (257° Fahr.) and weighed. The filtrate containing the bitumen is evaporated, the bituminous residue burned, and the weight of the ash thus obtained added to that of the residue in the two flasks and the crucible. The sum of these weights deducted from the weight of substance taken gives the weight of bitumen extracted.

Solubility of Bitumen in Carbon Tetrachloride (CCl_4).

The test shall be conducted in exactly the same manner as described for the test for "Solubility in Carbon Disulphide", except that 100 cu. cm. (3.381 oz.) of chemically pure carbon tetrachloride shall be used in place of carbon disulphide, and the percentage of bitumen insoluble in carbon tetrachloride shall be reported on the basis of the bitumen taken as 100, the quantity of bitumen having been determined by the method described under the heading "Solubility in Carbon Disulphide."

Consistency.

The "Engler Viscosimeter," the "New York Testing Laboratory Float," or the "Penetrometer," shall be used, as practicable, at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), 46° cent. (115° Fahr.), and 98° cent. (208° Fahr.).

Viscosity Test.

The viscosity of liquid bituminous materials shall be determined at any desired temperature by using the "Engler Viscosimeter." This apparatus consists of a brass vessel for holding the material to be tested, and is closed by a cover. To the conical bottom is fitted a conical outflow tube exactly 20 mm. (0.787 in.) long, with a diameter of 2.9 mm. (0.114 in.) on top, and of 2.8 mm. (0.110 in.) on the bottom. This tube is closed and opened by a pointed hardwood stopper. Pointed metal projections are placed on the inside of the vessel at equal distances from the bottom, and serve for measuring the charge of material, which is 240 cu. cm. (8.116 oz.). A thermometer is used to ascertain the temperature of the material to be tested. The vessel is surrounded by a brass jacket, which holds the material which may be used as a heating bath, either water or

cottonseed oil, according to the temperature at which the test is to be made. A tripod serves as a support for the apparatus, and also carries a ring burner by which the bath is heated directly. The measuring cylinder, having a capacity of 100 cu. cm. (3.381 oz.), which is sufficiently accurate for work with road materials, is placed directly under the outflow tube.

As all viscosity determinations should be compared with that of water at 25° cent. (77° Fahr.), the apparatus shall have been previously calibrated as follows: The cup and outlet tube shall first be scrupulously cleaned. A piece of soft tissue paper is convenient for cleaning the tube. The stopper shall then be inserted in the tube, and the cup shall be filled with water at 25° cent. (77° Fahr.) to the top of the projections. The measuring cylinder shall be placed directly under the outflow tube so that the material, on flowing out, will not touch the sides. The stopper shall then be removed and the time required, both for 50 and 100 cu. cm. (1.691 and 3.381 oz.) to run out, shall be ascertained by using a stop-watch. The results thus obtained shall be checked a number of times. The time required for 50 cu. cm. (1.691 oz.) of water should be about 11 sec. and for 100 cu. cm. (3.381 oz.) about 22.8 sec.

Bituminous materials shall be tested in the same manner as water, and the temperature at which the test is made shall be controlled by the bath. The material shall be brought to the desired temperature and maintained there for at least 3 min. before making the test. The results are expressed as specific viscosity compared with water at 25° cent. (77° Fahr.), as follows:

$$\text{Specific viscosity at —— degrees centigrade for —— cu. cm.} = \frac{\text{seconds for passage of given volume at —— degrees centigrade}}{\text{seconds for passage of same volume of water at 25° cent. (77° Fahr.)}}.$$

Float Test.

The float apparatus consists of two parts, an aluminum float or saucer and a conical brass collar. The two parts are made separately, so that one float may be used with a number of brass collars.

In making the test, the brass collar shall be placed with the small end down on the brass plate, which shall have been previously amalgamated with mercury by rubbing it first with a dilute solution of mercuric chloride or nitrate and then with mercury. A small quantity of the material to be tested shall be heated in the metal spoon until quite fluid, with care that it shall suffer no appreciable loss by volatilization and that it shall be kept free from air bubbles. It shall then be poured into the collar in a thin stream until slightly more than level with the top. After the material has cooled to room temperature, the surplus may be removed with a spatula blade which

has been slightly heated. The collar and plate shall then be placed in one of the tin cups containing ice water maintained at 5° cent. (41° Fahr.), and left in this bath for 15 min. Meanwhile, the other cup shall be filled about three-fourths full of water and placed on the tripod, and the water shall be heated to any temperature desired for the test. This temperature shall be accurately maintained, and shall at no time throughout the entire test be allowed to vary more than 0.5° cent. (0.9° Fahr.) from the temperature selected. After the material to be tested has been kept in the ice water for 15 min., the collar and contents shall be removed from the plate and screwed into the aluminum float, which shall then be immediately floated in the warmed bath. As the plug of bituminous material becomes warm and fluid, it is gradually forced upward and out of the collar, until water gains entrance to the saucer and causes it to sink.

The time, in seconds, between placing the apparatus on the water and when the float sinks shall be taken as a measure of the consistency of the material under examination.

Penetration Test.

The penetration test for determining the consistency of a material shall be made by measuring the distance a weighted needle will penetrate into the material at a given temperature. A standard No. 2 cambric needle, weighted with 100 grammes, shall be used, except as noted below, and the depth of penetration shall be determined on the bituminous material maintained at 25° cent. (77° Fahr.) while the load is applied for 5 sec. When the penetration test, made as above described, will give a result of less than 10 or more than 350, the loading and time shall be changed to 200 grammes for 1 min., and 50 grammes for 5 sec., respectively. The penetration test shall also be made on the bituminous material under the following conditions whenever practicable: Penetration at 4° cent. (39° Fahr.) with a weight of 200 grammes for 1 min.; penetration at 46° cent. (115° Fahr.) with a weight of 50 grammes for 5 sec. The loading. time, and temperature shall be reported, with the resulting penetration in every case. The unit of penetration shall be reported as 0.1 mm.

The penetration test is made as follows:

The sample of the material to be tested shall first be warmed just sufficiently to flow freely, stirred thoroughly, and poured into a flat-bottomed tin box, 5.715 cm. (2.25 in.) in diameter, and with sides about 3.810 cm. (1.5 in.) in height. The box and contents, after cooling in the air in the laboratory for 10 min., shall be immersed in ice water for 20 min., and shall then be placed in water maintained at the temperature at which the material is to be tested, and allowed to remain immersed for 2 hours, during which period the water containing the sample shall be maintained at the prescribed temperature

accurately and constantly. The sample, in a glass cup and covered with as much water, at the temperature at which the material is to be tested, as may be convenient without spilling, shall now be removed to the penetrometer. The needle shall have been firmly fixed in the machine and shall now be lowered with its holder until the point of the needle almost touches the surface of the sample. It shall then be cautiously brought down until the point of the needle just comes in contact with the surface of the sample; as will best be seen by having a light placed in such a way that, on looking through the sides of the glass cup, the needle will be reflected from the surface of the sample. After thus setting the needle, the rod connected with the recording dial shall be slowly lowered until the foot of the rod shall rest on the head of that part of the penetrometer carrying the needle, and a reading of the dial shall be taken, or the dial shall be adjusted to read at a convenient figure, such as 0 or 100.

Simultaneously noting the time and releasing the needle plunger, which shall have been previously weighted properly for the particular conditions above mentioned, the needle shall be allowed to act freely for exactly 5 sec., as determined by a pendulum or by a chronometer, when it shall be stopped. The rod shall then be lowered until it rests on the head of the part of the instrument carrying the needle, and the difference between the first and second readings of the dial shall be taken as the distance penetrated by the needle, or the penetration.

Owing to the susceptibility of certain bituminous materials to slight changes in temperature, the water bath must be maintained accurately at the desired temperature both before and during the test, and, if the room temperature differs greatly from that of the bath, the water in the glass cup shall be renewed from the bath after each trial. An average of from three to five trials, which should not differ from one another more than three units, shall be taken as the penetration of the sample.

The needle shall be cleaned thoroughly by wiping with a dry cloth before and after each trial. The point of the needle should be examined with a magnifying glass from time to time to see if it has become injured in any way. If it is found to be defective, the needle shall be replaced with a perfect one. Care shall be taken that the needle, in place in the machine and released for action, shall be loaded as required.

Melting Point.

The material under examination shall be first melted in a spoon by the gentle application of heat until sufficiently fluid to pour readily. Care shall be taken that it suffers no appreciable loss by volatilization. It shall then be poured into a 12.7-mm. (0.5-in.) brass cubical mould, which shall have been amalgamated with mercury and shall be placed

on an amalgamated brass plate. The brass may be amalgamated by washing it first with a dilute solution of mercuric chloride or nitrate, after which the mercury is rubbed into the surface. By this means the bituminous material is, to a considerable extent, prevented from sticking to the sides of the mould. The hot material shall slightly more than fill the mould, and, when cooled, the excess shall be cut off with a hot spatula.

After cooling to room temperature, the cube shall be removed from the mould and fastened on the lower arm of a No. 10 wire (B. & S. gauge), bent at right angles at one end and suspended beside a thermometer in a covered Jena glass beaker having a capacity of 400 cu. cm. (13.526 oz.), which shall be placed in a water bath, or, for high temperatures, a cottonseed-oil bath. The wire shall be passed through the center of two opposite faces of the cube, which shall then be suspended with its base 25.4 mm. (1 in.) above the bottom of the beaker. The water or oil bath shall consist of an 800-cu. cm. (27.051-oz.) low-form Jena glass beaker suitably mounted for the application of heat from below. The beaker in which the cube is suspended shall be of the tall-form Jena type, without lip. The metal cover shall have two openings. A cork, through which passes the long arm of the wire, shall be inserted in one hole and the thermometer in the other. The bulb of the thermometer shall be just level with the cube and at an equal distance from the side of the beaker. In order that a reading of the thermometer may be made, if necessary, at the point which passes through the cover, the hole shall be triangular and covered with an ordinary object glass through which the stem of the thermometer may be seen. Readings made through this glass shall be calibrated to the angle of observation, which may be made constant by sighting always from the front edge of the opening to any given point on the stem of the thermometer below the cover.

After the test specimen shall have been placed in the apparatus, the liquid in the outer vessel shall be heated in such a manner that the thermometer registers an increase of 5° cent. (9° Fahr.) per min. The temperature at which the bituminous material touches a piece of paper placed in the bottom of the beaker shall be taken as the melting point. Determinations made in the manner described shall not vary more than 2° cent. (3.6° Fahr.) for successive trials on the same material. At the beginning of this test the temperature of both bituminous material and bath shall be approximately at 25° cent. (77° Fahr.).

Loss on Evaporation.

Fifty grammes of the material shall be heated in a flat bottomed dish, 5.715 cm. (2.25 in.) in diameter and with sides about 3.810 cm. (1.5 in.) high for a total of 5 hours in three successive periods of 3, 1, and 1 hours, respectively, in a well-ventilated oven, the interior of

which shall be maintained at a uniform and constant temperature of 163° cent. (325° Fahr.). The oven temperature shall be controlled by any thermo-regulator effective within 2° cent. (3.6° Fahr.), and shall be brought to its full temperature before the material is introduced. The dish shall be level, and its contents, without removal from the oven, shall be stirred thoroughly for 1 min. between the successive periods mentioned. In preparing the residue for the penetration test, it shall first be heated, and thoroughly stirred while cooling.

Distillation.

(Method recommended by the Sub-Committee on Distillation of the American Society for Testing Materials. Note: Equivalents in English units, have been added by the Special Committee on Materials for Road Construction.)

Sampling.—The sample as received shall be thoroughly stirred and agitated, warming, if necessary, to insure a complete mixture before the portion for analysis is removed.

Dehydration.—If the presence of water is suspected, or known, the material shall be dehydrated before distillation. About 500 cu. cm. (16.907 oz.) of the material is placed in an 800-cu. cm. (27.051-oz.) copper still provided with a distilling head connected with a water-cooled condenser. A ring burner is used, starting with a small flame at the top of the still, and gradually lowering it, if necessary, until all the water has been driven off. The distillate is collected in a 200-cu. cm. separatory funnel with the tube cut off close to the stop-cock. When all the water has been driven over and the distillate has settled out, the water is drawn off and the oils are returned to the residue in the still. The contents of the still shall have cooled to below 100° cent. (212° Fahr.) before the oils are returned, and they shall be well stirred and mixed with the residue.

Apparatus.—The apparatus shall consist of the following standard parts:

(*a*) Flask.—The distillation flask shall be a 250-cu. cm. Engler distilling flask, having the following dimensions:

Diameter of bulb	8.0 cm.	(3.150 in.)
Length of neck	15.0 "	(5.906 ")
Diameter of neck	1.7 "	(0.670 ")
Surface of material to lower side of tubulature	11.0 "	(4.331 ")
Length of tubulature	15.0 "	(5.906 ")
Diameter of tubulature	0.9 "	(0.354 ")
Angle of tubulature	75°	

A variation of 3% from the foregoing measurements will be allowed.

(*b*) Thermometer.—The thermometer shall be of hardened glass, filled with carbon dioxide under pressure, and provided with an

expansion chamber at the top; it shall read to 450° cent. (842° Fahr.), shall be graduated in single degrees, centigrade, and shall have the following dimensions:

Diameter of stem....	6.75	to	7.25	mm.	(0.266	to	0.286	in.)
Length of thermometer	335	"	350	"	(13.189	"	13.780	")
Length from 0° to 450° marks	285	"	300	"	(11.221	"	11.811	")
Length of bulb.....	20	"	22	"	(0.787	"	0.866	")
Diameter of bulb....	5.25	"	6.50	"	(0.207	"	0.256	")

It shall rise from 15° to 95° cent. (59° to 203° Fahr.) in not less than 3 sec. nor more than 5 sec. when plunged into boiling water.

The thermometer shall be set up as for the distillation test, using water, napthalene, and dimethylamine as distilling liquids. The correctness of the thermometer shall be checked at 0° cent. (32° Fahr.) and 100° cent. (212° Fahr.) after each third distillation until seasoned.

(*c*) Condenser.—The condenser tube shall have the following dimensions:

Length			500	mm.	(19.685	in.)		
Width	12	to	15	"	(0.472	to	0.591	in.)
Width of adaptor end.....	20	"	25	"	(0.787	"	0.984	")

(*d*) Stands.—Two iron stands shall be provided, one with a universal clamp for holding the condenser, and one with a light grip arm with a cork-lined clamp for holding the flask.

(*e*) Burner and Shield.—A Bunsen burner shall be provided, with a tin shield, 20 cm. (7.784 in.) long and 9 cm. (3.543 in.) in diameter. The shield shall have a small hole through which to observe the flame.

(*f*) Cylinders.—The cylinders used in collecting the distillate shall have a capacity of 25 cu. cm. (0.845 oz.), and shall be graduated in tenths of a cubic centimeter.

Setting up the Apparatus.—The apparatus shall be set up, the thermometers being placed so that the top of the bulb is opposite the middle of the tubulature. All connections shall be tight.

Method.—One hundred cubic centimeters (3.381 oz.) of the dehydrated material to be tested shall be placed in a tared flask and weighed. After adjusting the thermometer, shield, condenser, etc., the distillation is commenced, the rate being regulated so that 1 cu. cm. (0.034 oz.) passes over every minute. The receiver is changed as the mercury column just passes the fractionating point.

			Up to 110°		cent.	(230°	Fahr.)	
110°	cent.	"	170°	"		(338°	"	)
170°	"	"	235°	"		(455°	"	)
235°	"	"	270°	"		(518°	"	)
270°	"	"	300°	"		(572°	"	)

To determine the quantity of residue, the flask is weighed again when distillation is complete. During the distillation the condenser tube shall be warmed when necessary, in order to prevent the deposition of any sublimate. The percentages of fraction should be reported, both by weight and by volume.

Ductility.

A briquette of the material to be tested shall be formed by pouring the molten material into a briquette mould. The dimensions of the briquette shall be: 1 cm. (0.394 in.) in thickness throughout its entire length; distance between the clips or end pieces, 3 cm. (1.181 in.); width of asphalt cement section at mouth of clips, 2 cm. (0.787 in.); width at minimum cross-section, half way between clips, 1 cm. (0.394 in.). The center pieces are removable, the briquette mould being held together during moulding with a clamp or wire.

The moulding of the briquette shall be done as follows: The two center sections shall be well amalgamated to prevent the asphalt cement from adhering to them, and the briquette mould shall then be placed on a freshly amalgamated brass plate. The asphalt cement to be tested, while in a molten state, shall be poured into the mould, a slight excess being added to allow for shrinkage on cooling. When the asphalt cement in the mould is nearly cool, the briquette shall be cut off level, with a warm knife or spatula. When it is thoroughly cooled to the temperature at which it is desired to make the test, the clamp and the two side pieces are removed, leaving the briquette of asphalt cement held at each end by the ends of the mould, which now play the part of clips. The briquette shall be kept in water for 30 min. at 4° cent. (39° Fahr.) or 25° cent. (77° Fahr.) before testing, dependent on the temperature at which the ductility is desired. The briquette with the clips attached shall then be placed in a "ductility test machine" filled with water at one of the above temperatures to a sufficient height to cover the briquette not less than 50 mm. (1.969 in.). This machine consists of a rectangular water-tight box, having a movable block working on a worm-gear from left to right. The left clip is held rigid by placing its ring over a short metal peg provided for this purpose; the right clip is placed over a similar rigid peg on the movable block. The movable block is provided with a pointer which moves along a centimeter scale. Before starting the test, the centimeter scale is adjusted to the pointer at zero. Power is then applied by the worm-gear pulling from left to right at the uniform rate of 5 cm. (1.969 in.) per min. The distance, in centimeters, registered by the pointer on the scale at the time of rupture of the thread of asphalt cement shall be taken as the ductility of the asphalt cement.

Solubility in Petroleum Naphtha, and Character of Residue on Glass.

Two grammes of the material shall be placed in a 4-oz. oil-sample bottle, made up to 100 cu. cm. (3.381 oz.) with 88° Baumé petroleum naphtha (boiling point between 40° cent. (104° Fahr.) and 55° cent. (131° Fahr.), and the whole well shaken until the sample is digested. The bottle shall then be centrifugalized for 10 min., 50 cu. cm. (1.691 oz.) withdrawn into a weighed flask, the naphtha distilled off by a water bath, and the residue weighed. From this weight the percentage of solubility shall be calculated.

Although there is no generally accepted method of determining the stickiness or oiliness of bituminous materials, except by the unsatisfactory method of handling, a test which has been used to a limited extent, and is based on the character of the residue on glass, appears to the Committee to give promise. This test is made as follows:

One-half gramme of the residue obtained as just specified shall then be placed in the center of a circular glass disk, 32.258 sq. cm. (5 sq. in.) in area, and a similar disk shall be imposed upon it. By warming and pressure of the fingers the material shall be as evenly distributed as practicable over the adjacent surfaces of the disks, and the disks with the material between them shall then be stored under a weight of 1 kg. and at a temperature of 25° cent. (77° Fahr.) for at least 2 hours. The disks shall then be pulled apart horizontally and in a direction truly parallel to their facial diameters at a regular rate of movement of 2.540 cm. (1 in.) per min., the maximum stress, in grammes, required being recorded and reported.

Fixed Carbon.

One gramme of the bituminous material shall be placed in a platinum crucible weighing between 20 and 30 grammes, between 28 and 38 mm. (1.102 and 1.496 in.) in height, and having a tightly fitting cover provided with a flange about 4 mm. (0.157 in.) in depth. The crucible and its contents shall then be heated, first gently and then more severely, until no smoke or flame shall issue between the crucible and the lid. It shall then be placed in the full flame of a Bunsen burner for 7 min., holding the cover down with the end of a pair of tongs until the most volatile products shall have been burned off. The crucible shall be supported on a platinum triangle with the bottom 6 to 8 cm. (2.362 to 3.150 in.) above the top of the burner. The flame shall be fully 20 cm. (7.874 in.) high when burning free, and the determination shall be made in a place free from drafts. The upper surface of the cover shall burn clear, but the under surface may or may not be covered with carbon, dependent on the character of the bituminous material. The crucible shall be removed to the desicca-

tor, and, when cool, shall be weighed, after which the cover shall be removed and the crucible placed in an inclined position over the Bunsen burner and ignited until nothing but ash remains. Any carbon deposited on the cover shall also be burned off. The weight of ash remaining shall be deducted from the weight of the residue after the first ignition of the sample. The resulting weight is that of the fixed carbon, which shall be calculated on the basis of the total weight of the sample, exclusive of mineral matter.

Paraffin.

One hundred grammes of the material shall be distilled rapidly in a retort to a dry coke. Five grammes of the distillate shall then be thoroughly mixed in a 60-cu. cm. (2.029-oz.) flask with 25 cu. cm. (0.845 oz.) of Squibbs' absolute ether. Twenty-five cu. cm. (0.845 oz.) of Squibbs' absolute alcohol shall then be added, and the flask packed closely in a freezing mixture of finely crushed ice and salt for at least 30 min. The precipitate shall be filtered out quickly with a suction pump, using a No. 575 C. S. and S. 9-cm. hardened filter paper. The flask and precipitate shall then be rinsed and washed with a mixture of equal parts of Squibbs' alcohol and ether cooled to — 17° cent. (1° Fahr.) until free from oil (50 cu. cm. (1.691 oz.) of washing solution is usually sufficient). When sucked dry, the filter paper shall be removed and the waxy precipitate transferred to a small glass disk and evaporated on a steam bath. The residue (paraffin) remaining on the disk shall be weighed, and from this weight the percentage on the original 5-gramme sample shall be calculated.

(Note: Although the foregoing method is the one in general use, the following modification is believed by the Committee to promise accurate results and to merit thorough trial.)

One hundred grammes of the material shall be distilled rapidly in a retort to dry coke. The distillate shall then be thoroughly mixed. A 5-gramme sample of the distillate shall be placed in a tared 60-cu. cm. (2.029-oz.) flask with 25 cu. cm. (0.845 oz.) of Squibbs' ether. Twenty-five cubic centimeters (0.845 oz.) of Squibbs' alcohol shall be added, after which the mixture shall be shaken well and the flask packed closely in a freezing mixture of finely crushed ice and salt for at least 30 min. Without removing the flask from the freezing mixture, the solvent shall be sucked from the precipitated paraffin by using a tube connected with the filter pump, the end of the tube being covered with an accurately weighed tuft of absorbent cotton. When the solvent is all removed, the flask shall be taken from the freezing mixture and the cotton tuft pushed off the tube into the flask. The waxy precipitate shall be dissolved, with the tuft of cotton, in 25 cu. cm. (0.845 oz.) of Squibbs' absolute ether. Twenty-five cubic centimeters (0.845 oz.) of

Squibbs' absolute alcohol shall then be added and the flask placed in the freezing mixture again for 30 min. The solvent shall then be removed with the pump and tube as described previously, keeping the flask in the freezing mixture. The second cotton tuft shall be pushed into the flask. The flask and contents shall then be placed in the drying oven at 100° cent. (212° Fahr.) until constant weight shall be reached. This weight, less the weight of the cotton used and less the weight of the tared flask, is the weight of the paraffin. Calculate the weight of the paraffin to a percentage of the original material.

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PAPERS AND DISCUSSIONS

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PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS.*

To the American Society of Civil Engineers:

The Special Committee "to consider and report upon the design, ultimate strength, and safe working values of steel columns and struts", presents the following report of progress.

Some seventy columns on the programme of tests, which is being made for the Society by the Bureau of Standards, U. S. Government, S. W. Stratton, Director, have been tested. All the columns for both the light and heavy series have been fabricated and delivered at Washington.

During the past year, the Committee has held five regular meetings, most of these being called at Washington, in order to keep in close touch with the test programme, and a number of sub-committee meetings to consider special details.

Through W. H. Moore, M. Am. Soc. C. E., Chairman of the Sub-Committee on Built-up Columns of the Committee on Iron and Steel Structures of the American Railway Engineering Association, your Committee has kept in touch with their programme of tests.

The Committee has spent considerable time studying the tests that have been made, and has been able to suggest variations in methods which give additional information and improve the value of the records.

The methods of testing have been fairly well outlined, and as your Committee's final report will no doubt give emphasis to results rather than to methods, it is deemed wise at this time to report quite fully on this latter feature, in order that the members of the Society may become familiar with what is being done, and may be able to make suggestions to the Committee covering the interpretation of the results.

Testing Machine and Instruments.—All the tests have been made at the Bureau of Standards, with an Emery testing machine, which

* To be presented to the Annual Meeting, January 20th, 1915.

was authorized by Congress on March 4th, 1909, and was accepted in 1913. Its capacity is as follows:

Tension	1 150 000 lb.
Compression	2 300 000 lb.
Extreme length between heads:	
Tension	34 ft. 4 in.
Compression	33 ft. 1¾ in.
Clearance between screws	4 ft. 6 in.
Length of stroke of ram	5 ft.

It will be seen from Fig. 1 that it has two heads, one fixed and the other movable, on two straining screws, each 12 in. in diameter. The hydraulic ram which furnishes the pressure is in the movable head, and, as the stroke is only 5 ft., it is necessary to adjust this head for columns of different lengths. The weighing is done from the fixed head of the machine.

The weighing device, which is peculiar to the Emery machines, consists of a specially constructed hydraulic support in the fixed head of the machine, which is connected with a smaller hydraulic chamber in the scale case. The load on the test specimen is transmitted directly to the oil in the hydraulic support, and by hydrostatic pressure is transmitted to the hydraulic chamber which actuates the levers of the weighing scale. These levers are all of the plate-fulcra type, characteristic of all Emery testing machines.

The construction of the hydraulic support and chamber is such that a movement of 0.01 in. of the brass diaphragms is sufficient to register the full capacity of the machine. By use of these diaphragms and guiding rings, friction of the parts is eliminated.

The programme calls for all the tests to be made with square-ended columns, and this is done by providing cast-steel bearing blocks at each head, which bear directly against the columns. Considerable difficulty has been found in securing bearing blocks of sufficient accuracy, and it was necessary for the Bureau to go over all the blocks and dress them to a surface with a variation of not more than plus or minus 0.001 in.

For measuring the compressions on the longitudinal axis of the columns, compressometers were used, which consisted of a bar, fixed at one end to the column, and the other end connected with a micrometer gauge, which reads by estimate to ten-thousandths of an inch. This gauge was equipped with an electrical buzzer, which enabled the operator always to secure an equal contact. At the beginning of the series, wood bars were used, but, owing to the length of time taken in making a test, it was decided to substitute hollow steel bars, in order to avoid any possibility of changes due to moisture and temperature.

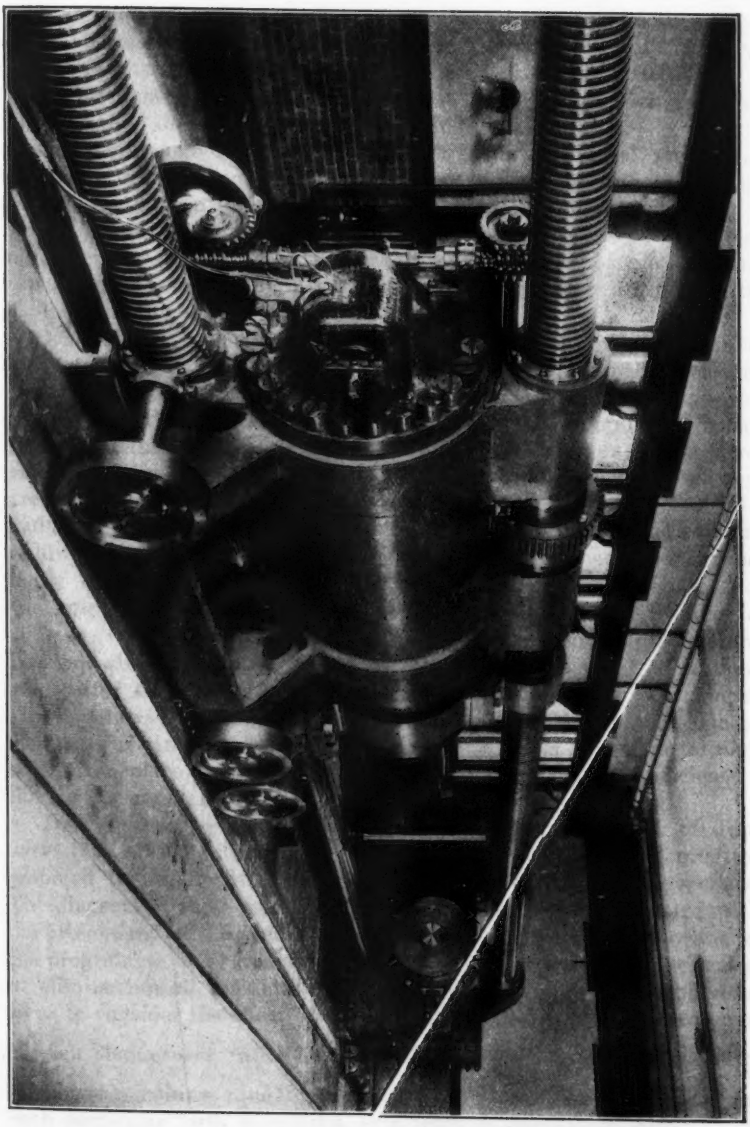


FIG. 1.—2 300 000-POUND TESTING MACHINE, BUREAU OF STANDARDS, WASHINGTON, D. C.

The local stresses were measured with a Berry strain gauge, which is made with a pair of Invar steel bars some 10 in. long, with a fixed point at one end and a movable point with a bell-crank at the other end. The horizontal arm is connected with an Ames dial gauge, which reads, with the multiplication of the bell crank, to 0.0002 in., and by estimating to 0.00002 in. The nominal length between the contact points of this strain gauge is 8 in.

The strain gauge measurement was made in the usual way, *viz.*, two holes were drilled, 8 in. apart, using a No. 55 drill, and then countersunk with a tool having an angle of about 110 degrees. This insures a good shoulder for the points on the strain gauge, which is highly essential for accurate work with this type of instrument. Experienced operators can secure very uniform readings within limits of plus or minus 0.0002 in.

Originally, the horizontal and vertical deflections of the column under test were measured with a piano wire, with a spring tension which was fastened to plugs set in each end of the column. Measurements were taken at the center by scales. It was found that when the ends of the column began to deflect, or warp, the local bends were transmitted to the wire, and this method was then improved by stretching the wire from a fixed frame below the column. The difficulty in securing accurate measurements from the wire to the column within limits of plus or minus 0.01 in. still remained, and it was finally overcome by abandoning the longitudinal wire entirely, and connecting the column at the center with two dial gauges reading to 0.001 in., which were supported on frames, independent of the column. Tension wires perpendicular to the longitudinal axis of the column transmitted the horizontal and vertical movements to the two gauges. Two operators were employed in manipulating the gauges, and, due to the length of time required to read these, some slight flow of metal may have taken place during the reading, but this was hardly sufficient to change the general result.

Test Columns.—The Progress Report of the Committee, of January 15th, 1913,* shows the column sections which the Committee proposed to use, together with the chemical and physical properties. The diagram of these sections and the properties are incorporated with the present report. In addition, it should be noted from this report that the programme was "arranged with the intention, as far as practicable, of eliminating all variables except that of form", and was subdivided so as to consider the effect of light and heavy sections. Three columns of each slenderness ratio, 50, 85, and 120 of $\frac{L}{R}$, were provided for all shapes, making nine light sections and nine heavy sections, for each particular form of column, or a total of 144 test columns.

* *Proceedings*, Am. Soc. C. E., for February, 1913.

In addition to these, the Bureau of Standards has provided eighteen columns of Bethlehem H-sections, and eighteen columns of Carnegie 6-in. and 8-in. H-sections.

TABLE OF CHEMICAL AND PHYSICAL PROPERTIES FOR SPECIAL SPECIFICATIONS FOR UNIFORM QUALITY STRUCTURAL STEEL FOR COMPARATIVE COLUMN TESTS FOR THE BUREAU OF STANDARDS, U. S. GOVERNMENT.

Material.—Basic open-hearth structural steel.

Chemical.—Uniformity is paramount, and must be aimed for.

Carbon.—The percentage of carbon must not vary more than just to insure the desired physical properties in required sections of material.

Manganese	0.40% min. and 0.50% max.
Phosphorus	0.00% min. and 0.03% max.
Sulphur	0.00% min. and 0.04% max.
Silicon	0.00% min. and 0.10% max.
Nickel	0.00% min. and 0.05% max.
Chromium	0.00% min. and 0.05% max.
Copper	0.00% min. and 0.03% max.
All possible others	0.00%

Physical.—The desired physical properties must be aimed for.

Ultimate tensile strength	Desired, 60 000 lb. per sq. in. (may vary 1 500 lb. either way).
Elastic limit (yield point)	Desired, 38 000 lb. per sq. in. (may vary 1 000 lb. either way).
Elongation in 8 in.	Desired, 28% (may vary 2% either way).
Reduction in area	Desired, 56% (may vary 4% either way).
Character of fracture	Silky, cup.
Cold bend without fracture	180°, flat, on itself.

During the rolling of the material at the mill, and the fabrication at the shop, the inspectors of the Bureau of Standards investigated the various processes, making notes of the different conditions to which the material was subjected. Care was taken that the material met the rigid specifications, and although some variations were allowed, it was only with the special permission of the Committee.

In addition to the material required for building the columns, the Bureau of Standards ordered additional pieces, 5 ft. long, of each shape and from each melt, so as to have an extra lot of unworked material on hand for special physical and chemical tests. The physical and chemical tests were first made on all shapes by the manufacturer, and

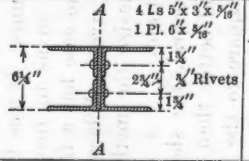
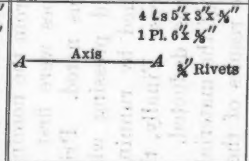
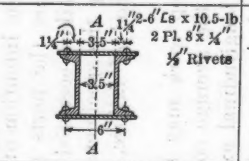
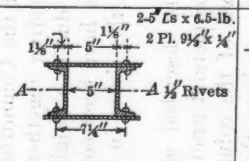
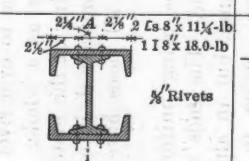
Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches	r	$\frac{l}{r}$	Length.
 4 Ls 5″ x 3″ x 5/16″ 1 Pl. 6″ x 5/16″	9.00 1.875 11.475	2.24	120 85 50	22′4 13/16″ 15′10 7/16″ 9′4″	 4 Ls 5″ x 3″ x ⅜″ 1 Pl. 6″ x ⅜″ ⅞″ Rivets	18.44 3.75 22.19	2.36	120 85 50	23′7 1/16″ 16′8 5/8″ 9′10″
 2-6″ Ls x 10.5-lb. 2 Pl. 8″ x ¼″ ⅞″ Rivets	6.18 4.00 10.18	2.31	120 85 50	23′1 3/16″ 16′4 3/8″ 9′7 1/2″	2-6″ Ls x 15.5-lb. 2 Pl. 8″ x ⅜″ ⅞″ Rivets	9.12 8.00 17.12	2.33	120 85 50	23′3 5/8″ 16′6 1/16″ 9′3 1/2″
 2-5″ Ls x 6.5-lb. 2 Pl. 9½″ x ⅛″ ⅞″ Rivets	3.90 4.75 8.65	2.34	120 85 50	23′4 13/16″ 16′6 1/16″ 9′9″	2 Ls 5″ x 11.5-lb. 2 Pl. 9½″ x ⅛″ ⅞″ Rivets	6.76 9.50 16.26	2.39	120 85 50	23′10 13/16″ 16′11 1/16″ 9′11 3/8″
 2 Ls 8″ x 11¼-lb. 1 I 8″ x 18.0-lb ⅞″ Rivets	6.70 5.33 12.03	2.38	120 85 50	23′9 3/8″ 16′10 5/16″ 9′11″	2 Ls 8″ x 18.75-lb. 1 I 8″ x 20.50-lb. ⅞″ Rivets	11.02 6.03 17.05	2.32	120 85 50	23′2 1/16″ 16′5 1/16″ 9′8″

FIG. 2.

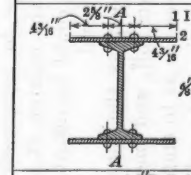
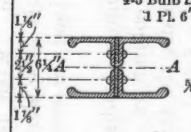
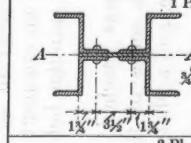
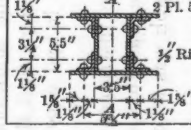
were then checked at the laboratory of the Bureau of Standards, at Pittsburgh.

It will be noted from the diagrams (Figs. 2 and 3) showing the cross-sections of the columns that the rivets were all of standard size, the smallest being ½ in. and the largest ¾ in. in diameter, and the rivet spacing called for four diameters throughout the total length of the columns. It was the intention of the Committee to over-rivet the sections, so as to insure the parts acting together as a whole. So far, in no case has a failure occurred due to a bulging of the material between rivets. Where the specimens have been compressed, in order to emphasize the method of failure, the rivets in all cases have remained tight.

In general, the shop work may be described as being uniformly good, and better than the average commercial grade. The ends were dressed square to the longitudinal axis of the column, but in almost all cases it was found necessary to re-dress them, owing to the fact that the ordinary commercial milling machine gives too rough a cut to insure complete bearing over the whole surface of the end cross-section of the column. After the test column was placed in the machine, special attention was given to insure a uniform bearing by applying loads of about 1 000 lb. per sq. in.

The investigation of the test columns developed variations in their sectional areas, these amounting, in extreme cases, to as much as 4% from the nominal area. In order to determine the correct area, the shapes were first calipered at the ends of the columns, and the variations noted. Prints of both ends were taken by blacking the ends and pressing white paper against them. The area was then determined by running a planimeter around the inside and outside perimeters. Finally, the column was weighed, the weight of the rivet heads was deducted, and the area was determined by the specific gravity of the material. This area, after being compared and checked with the results of the other two methods, was adopted as the correct one to be used in determining the unit loadings. When the column was put in the machine, before being tested, it was carefully examined for loose rivets, scale, or any other conditions which might affect its strength, and, during progress of the tests, rivets were tried and any scaling was noted. After the column was placed in the machine, a nominal load, of sufficient amount to insure its being held to the proper bearings, was applied, and the compressometers were attached. For the $\frac{L}{R}$ ratio of 50, 85, and 120, the test pieces ran on an average of 10, 16, and 23 ft. long, and the compressometers were arranged respectively with bars 50, 100, and 150 in. long. The two tests given* will indicate the details of applying the loads, and, as stated before,

*The details of these tests will be given later.

Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.
 1 I 10″ x 25-lb. 2 Pl. 11″ x 5/16″ ⅝″ Rivets	7.37 6.88 14.25	2.31	120 85 50	23′1³⁄₁₆″ 16′4³⁄₁₆″ 9′7³⁄₁₆″	1 I 10″ x 35-lb. 2 Pl. 11″ x ⅝″ ⅝″ Rivets	10.29 13.75 24.04	2.47	120 85 50	24′8³⁄₁₆″ 17′5¹⁵⁄₁₆″ 10′3¹⁄₂″
 4-5″ Bulb l³ x 10.1-lb. 1 Pl. 6″ x 5/16″ ⅝″ Rivets	11.88 1.88 13.76	2.48	120 85 50	24′0⅝″ 17′0¹³⁄₁₆″ 10′1″	4-5″ Bulb l³ x 13.5-lb. 1 Pl. 6″ x 5/16″ ⅝″ Rivets	15.88 2.63 18.51	2.42	120 85 50	24′2³⁄₁₆″ 17′1¹¹⁄₁₆″ 10′1″
 4-4″ Z's x ¼″ 1 Pl. 7″ x ¼″ ¾″ Rivets	9.61 1.75 11.39	2.46	120 85 50	24′7³⁄₁₆″ 17′5³⁄₁₆″ 10′3″	4-4″ Z's x ⅜″ 1 Pl. 7″ x ⅜″ ⅜″ Rivets	22.20 4.38 26.58	2.51	120 85 50	25′1³⁄₁₆″ 17′9³⁄₁₆″ 10′5¹⁄₂″
 2 Pl. 9″ x ¼″ 4 L s 2″ x 2″ x ¼″ 2 Pl. 5¼″ x ¼″ ⅜″ Rivets	4.500 3.760 2.625 10.885	2.33	120 85 50	23′3⅝″ 16′6¾₁₆″ 9′8½″	2 Pl. 9″ x 7/16″ 4 L s 2″ x 2″ x 7/16″ 2 Pl. 5¼″ x 7/16″ ⅜″ Rivets	7.88 6.21 4.59 18.71	2.34	120 85 50	23′4¹³⁄₁₆″ 16′6⅜″ 9′9″

General Notes:
Rivet Spacing = 4 diameters of rivet
End distance = 2 " " "
Make three columns of each length for all shapes

FIG. 3.

after each increment of 1 000 lb. per sq. in. had been applied, sufficient time was given to read the various strain gauges.

Failure of the column was first indicated by the dial gauges, which registered a rapid increase of the deflection at the center, and by the unbalancing of the scale beam, which showed a falling off of resistance to compression. With the exception of some additional scaling, the column at this time showed no apparent change, and the pressure could be maintained for several minutes before it had a perceptible bend.

As soon as the maximum loading was reached, the instruments were taken off, and one column of each series of three was further compressed, so as to emphasize the failure, and a photograph was taken to indicate how the column failed.

Methods of Testing.—After the material was ready for the beginning of the programme of testing, a conference was held between the physicists of the Bureau of Standards and a Sub-Committee of the Column Committee as to the methods of applying the loads and measuring the strains. A general outline of the instructions to the Engineer in Charge was agreed upon, and this outline is given below. Thus far it has not been necessary to make any marked changes in the methods. On certain tests, variations have been introduced, at the request of the Committee, or at the suggestion of the physicist, but, thus far, these have not produced enough variation from the general result to warrant their continuance.

Outline of Instructions for Testing.

Columns shall be weighed and gauged for sizes before being tested. In addition, inspection shall be made to determine the condition of the riveting, in order to see that the pieces are properly drawn together, especial note being made of any local conditions where the riveting has failed to do this. Observations should also be recorded where the pieces at the ends of the members have been bent. General alignment, waves in pieces or in the column as a whole, kinks, or any other special condition should be noted.

The ends of the columns shall be dressed flat to reduce the amount of shimming to a minimum.

The two longer series of columns, that is, those of 85, and $120\frac{L}{R}$ shall be counterweighted at the middle by an amount equal to one-half of the total weight of the column.

Strain gauge measurements along the length of the column, at points from 8 to 10 in. apart, shall be made at the center of the column and near both ends. The gauge points at the ends shall be placed far enough away to avoid effects of local distortion due to the taking up of the stresses. These strain gauge measurements shall be

made at each of the four corners of the column and at the center of the webs and flanges. Special shapes, such as the Z-bar and bulb angle shape columns, may require variations from this general rule in order to determine the variation in stress in the different parts of the column.

Compressometer measurements shall also be made on longer lengths, of from 50 to 150 in., in order to check the short strain gauge measurements. Four of these will be made on each column—two on the corners and one on the center of the flange, and one on the center of the web.

Deflection observations shall be made on the horizontal and vertical movements at the middle of the length of each test piece.

The condition of the cross-section of the column shall be determined by calipering the two open sides before applying the initial load, and at intervals of loading of 5 000 lb. per sq. in., in order to determine any change in the shape of the cross-section. This shall be done in as many places as may be necessary.

Initial loads of 1 000 lb. per sq. in. each shall be applied, and compression loads of 5 000, 10 000, and 15 000 lb. per sq. in. each, returning to the initial load and determining permanent sets; from 15 000 lb. proceeding with increments of 1 000 lb. per sq. in. each, determining permanent sets after loads of 20 000 and 25 000 lb. per sq. in. Also at 30 000 lb. per sq. in., if these several loads are reached before the column fails.

Horizontal and vertical deflections of the columns shall be observed at the time the longitudinal compressions and sets are measured. The general direction taken by the column when it fails, and whether due to deflection as a whole or local buckling, shall be noted.

Careful observations shall be made where the column first begins to scale, also the lines of scale. After failure, record the conditions of the rivets around the distortion points to see if any of these rivets are loose.

After the ultimate load has been reached, at least one column of each cross-section and length shall be compressed, to emphasize the manner of failure.

In general, photographs shall be taken of one column of each cross-section and length when the ultimate load has been reached, and in addition a photograph of each column on which the failure has been emphasized.

Examples of Tests.—The Committee gives the record of two of the tests, *Nos. 30 and 54, which contain the information as furnished by the Bureau of Standards.

A question arose as to the tendency of the material to flow under load, and in order to secure information on this point, a special time

* On account of the drawings required, the diagrams for these tests could not be given with the report; they will be reproduced later.

test was inaugurated. This is given as No. 54, where it will be seen that various loadings were applied and then held while the compressometers were read at intervals of 5 min. until the flow of the metal ceased. Although the results are interesting, and might be a matter of speculation in beginning a new programme, the variation in the failure point from similar columns does not seem to show enough difference to warrant any extensive carrying out of time tests in the present programme.

Considering the magnitude, both in the number of tests and the size of the columns, the present programme is being carried out with refinements which have never before been equalled. This is largely due to the enthusiastic co-operation of Dr. G. R. Olshausen, the Physicist in charge, and the cordial support of Dr. S. W. Stratton, the Director of the Bureau of Standards.

At the present time, less than one-half of the tests have been completed, and as it will be necessary to study the whole programme in order to correlate the results, the Committee makes this progress report.

AUSTIN LORD BOWMAN, *Chairman.* LEWIS D. RIGHTS, *Secretary.*

COMMITTEE:

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CHAS. F. LOWETH,
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RALPH MODJESKI,
FRANK C. OSBORN,
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PROGRESS REPORT OF THE SPECIAL COMMITTEE TO INVESTIGATE THE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS.

To the American Society of Civil Engineers:

The Special Committee appointed by the Board of Direction in January, 1913, "to investigate the conditions of employment of, and compensation of, civil engineers throughout the country; to collect statistics and data in reference thereto, and to report the results of such investigation to the Society", submitted a Progress Report to the Annual Meeting in January, 1914, and the Committee was continued.

Since the submission of this report the untimely death of its Chairman, Mr. Alfred Noble, whose loss was so generally deplored, has deprived the Committee of the advice of one whose knowledge and broad interest in the members of the Engineering Profession were of especial value in carrying out the work delegated to the Committee.

The Progress Report already submitted describes the methods adopted to secure information. These methods have been continued, but an effort has been made to reach a larger proportion of the membership of the Society. The Progress Report was based on information received from 3 638 members in various grades. On May 22d, 1914, a second circular was sent out, to which 1 240 replies were received; of these, 1 158 gave information which could be availed of, bringing the total number of usable replies to 4 796, which is two-thirds of the entire membership of the Society at the time the second circular was mailed. Inasmuch as it is impossible to identify those who have replied, there is no way in which the remaining members of the Society can be requested to furnish information, unless the entire membership is again circularized.

The information given in all the replies has been carefully analyzed under the direction of the Secretary, and has been incorporated

in a diagram, Plate XL, similar to that used in the first report of the Committee, and the new curves, while corresponding quite closely with those already published, are naturally more uniform and are believed to be more representative.

The irregularities in the curve representing the average yearly compensation have been generally reduced, although at some points they have been emphasized. For instance, although returns from engineers having 29 years of professional experience have been increased from 44 to 52, and although the maximum and minimum compensation remain the same, the eight additional replies have raised the average from $9 315 to $9 716, while at the next point on this curve, representing 30 years of active experience, although the maximum remains the same, there is a new minimum of $500, yet the average compensation has been increased from $5 281 to $5 346. Again, at 38 years, a new maximum, $10 000 greater that that formerly reported, has, with the same minimum, raised the average from $6 746 to $8 106; while at 39 years, where the maximum was and is $10 000, and the minimum $1 000, five additional replies have decreased the average from $4 600 to $4 329.

In the first report of the Committee reference was made to an attempt to draw a curve representing the compensation received by the middle man, but such a curve was not included in the diagram. Such a curve has been included in the diagram now submitted, Plate XL, and the Committee considers it worthy of special study.

Believing that the variation of compensation with the character of employment should be shown, a diagram, Fig. 1, has been prepared indicating the average compensation paid by the National Government, States, Municipalities, Technical Schools, Railroads, and Private Corporations and Companies, as well as the incomes of Consulting Engineers and Engineering Contractors. Inasmuch as the number of replies for each year of experience was relatively small, and might therefore be misleading, they have been grouped in 5-year periods, and the resulting curves are believed to be of interest and value. They indicate that the salaries paid by the National Government, States, Cities and Technical Schools, including in the last receipts from private practice, while closely corresponding with those paid by railroad companies during the first 20 years, are materially less after that period; that other corporations and private companies pay considerably higher salaries, and that the largest incomes are those of consulting and contracting engineers. That the curve showing the incomes of contracting engineers reaches its highest point at 30 years is probably due to the fact that it is only about 30 years ago that engineers became contractors to any considerable extent, or possibly to the fact that their success enables them to retire from active work at a relatively early age.

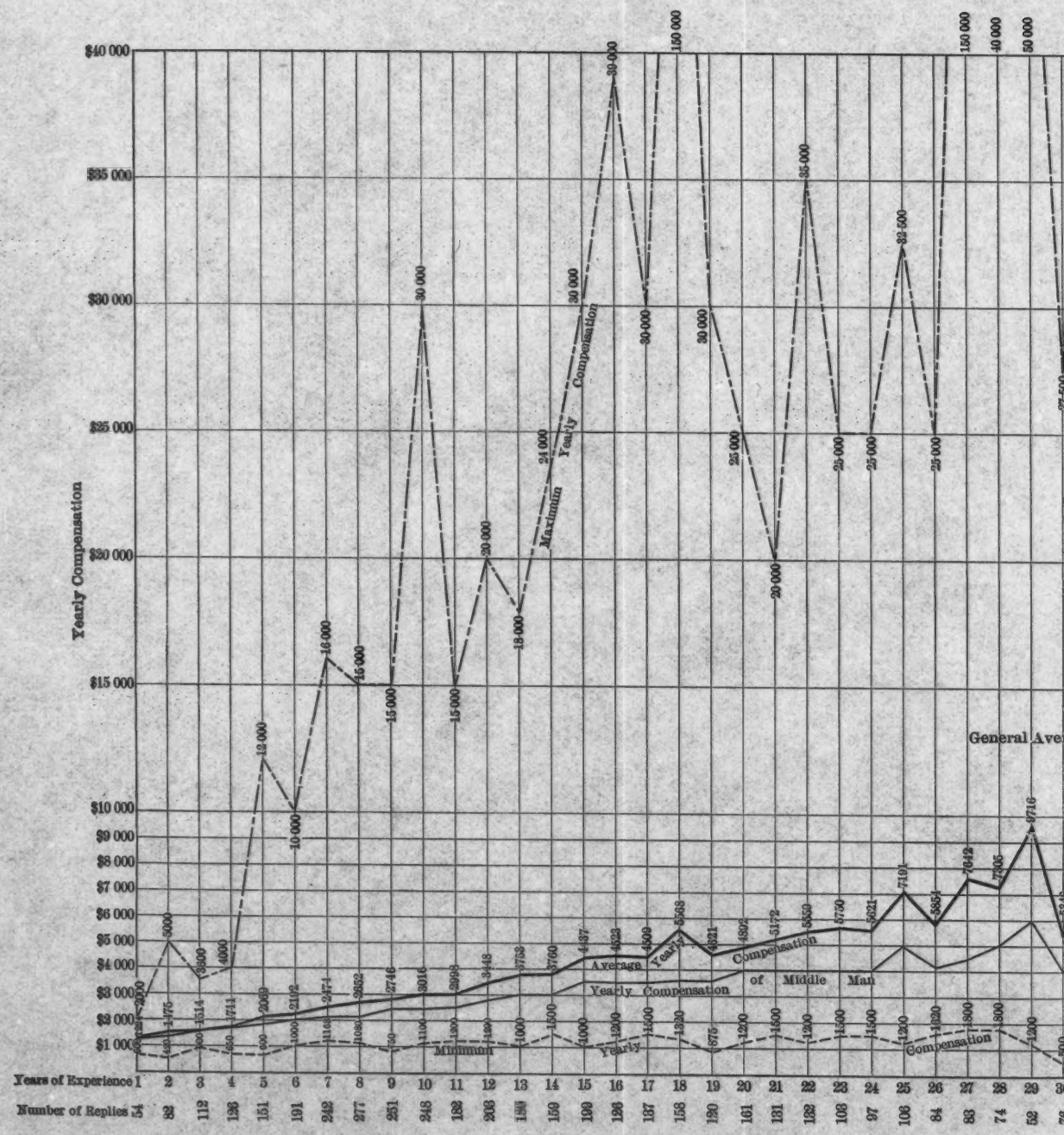
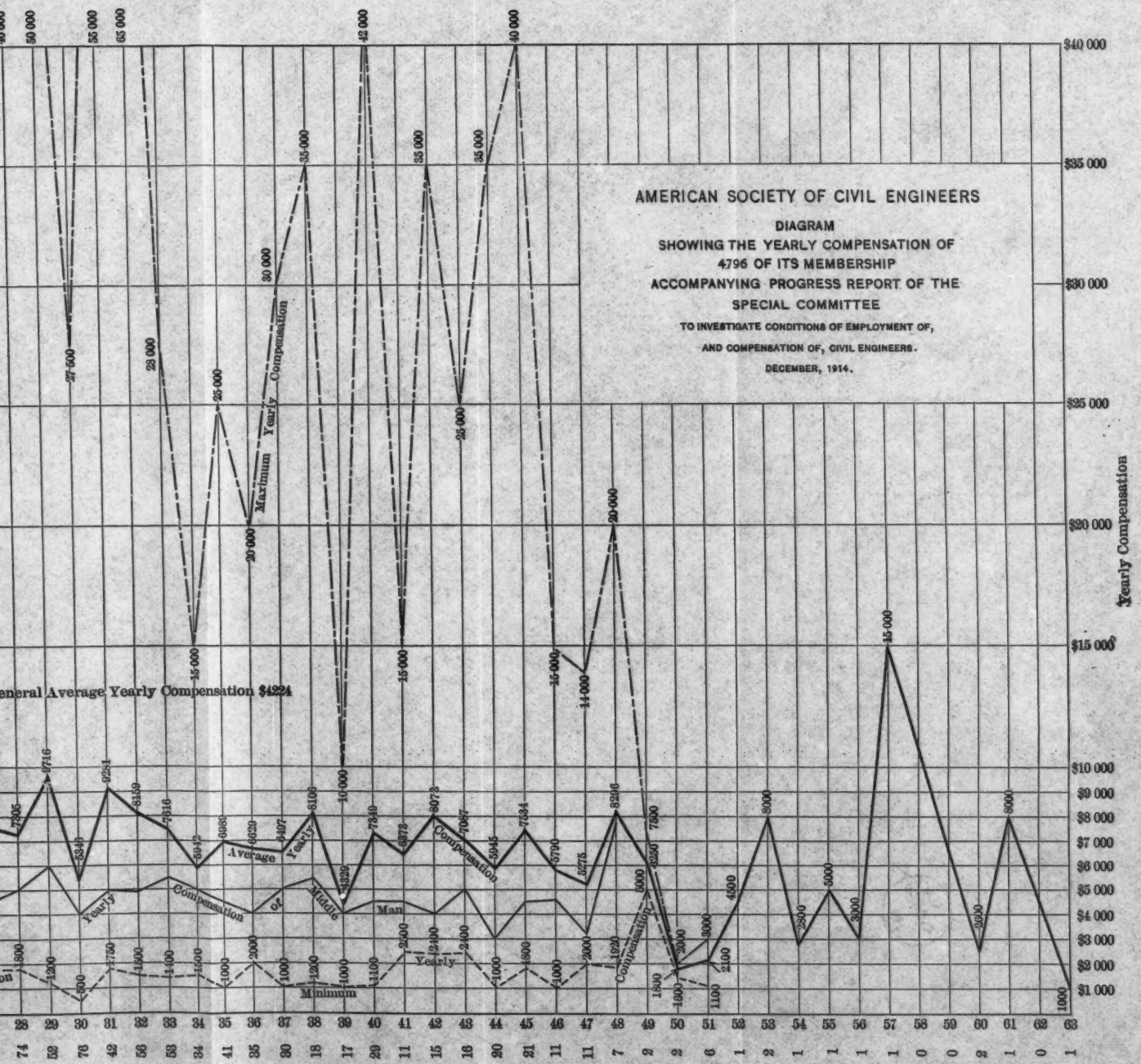
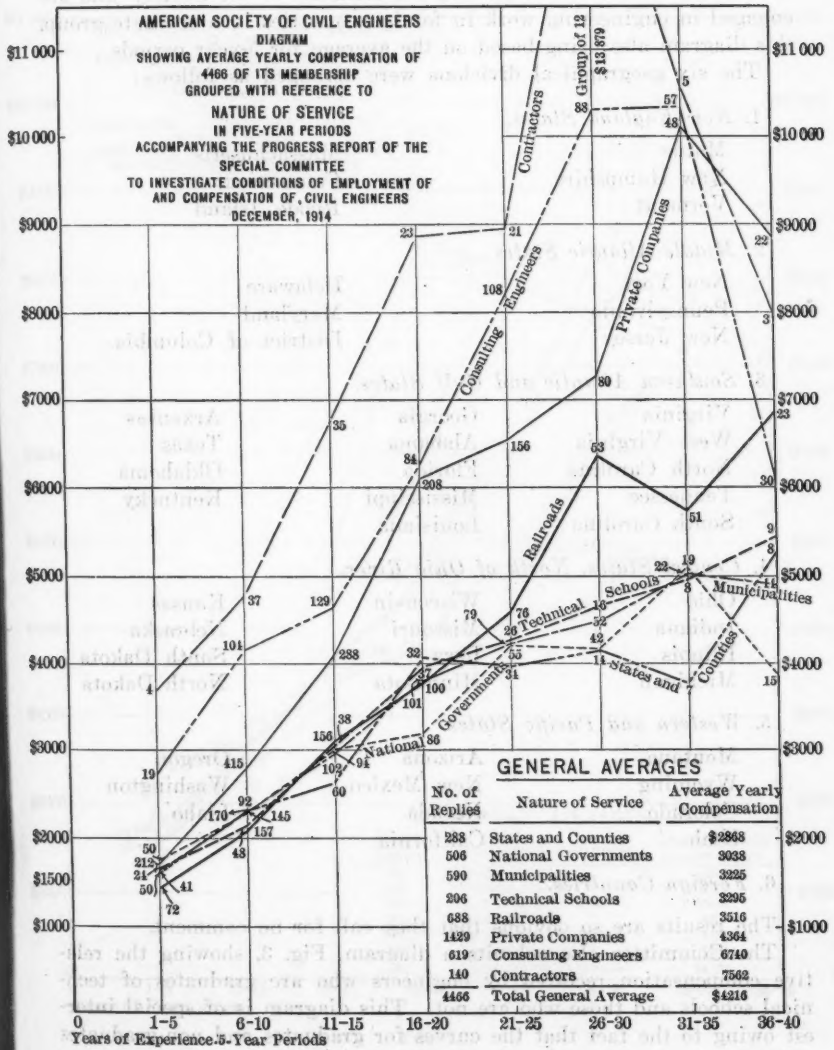

Yearly Compensation
$40 000
$35 000
$30 000
$25 000
$20 000
$15 000
$10 000
$9 000
$8 000
$7 000
$6 000
$5 000
$4 000
$3 000
$2 000
$1 000
Maximum Yearly Compensation
Average Yearly Compensation of Middle Man
Minimum Yearly Compensation
General Ave
Years of Experience 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29
Number of Replies 54 82 112 138 151 191 242 277 251 248 182 203 159 159 100 126 137 153 120 161 131 182 108 97 106 84 83 74 52

PLATE XL.
PAPERS, AM. SOC. C. E.
DECEMBER, 1914.
PROGRESS REPORT OF
SPECIAL COMMITTEE TO INVESTIGATE
THE CONDITIONS OF EMPLOYMENT OF, AND
COMPENSATION OF, CIVIL ENGINEERS.





GENERAL AVERAGES

No. of Replies	Nature of Service	Average Yearly Compensation
288	States and Counties	$2868
506	National Governments	3033
590	Municipalities	3225
206	Technical Schools	3295
688	Railroads	3516
1429	Private Companies	4364
619	Consulting Engineers	6740
140	Contractors	7562
4466	Total General Average	$4216

FIG. 1.

A diagram, Fig. 2, has been made showing the compensation received, arranged in geographical groups, using five different sections of the United States and placing the members of the Society who are engaged in engineering work in foreign countries in a separate group, this diagram also being based on the average for 5-year periods.

The six geographical divisions were arranged as follows:

1. *New England States.*

Maine	Massachusetts
New Hampshire	Connecticut
Vermont	Rhode Island

2. *Middle Atlantic States.*

New York	Delaware
Pennsylvania	Maryland
New Jersey	District of Columbia

3. *Southern Atlantic and Gulf States.*

Virginia	Georgia	Arkansas
West Virginia	Alabama	Texas
North Carolina	Florida	Oklahoma
Tennessee	Mississippi	Kentucky
South Carolina	Louisiana	

4. *Central States, North of Ohio River.*

Ohio	Wisconsin	Kansas
Indiana	Missouri	Nebraska
Illinois	Iowa	South Dakota
Michigan	Minnesota	North Dakota

5. *Western and Pacific States.*

Montana	Arizona	Oregon
Wyoming	New Mexico	Washington
Colorado	Nevada	Idaho
Utah	California	

6. *Foreign Countries.*

The results are so obvious that they call for no comment.

The Committee also submits a diagram, Fig. 3, showing the relative compensation received by engineers who are graduates of technical schools and those who are not. This diagram is of special interest owing to the fact that the curves for graduates and non-graduates follow each other very closely for the first 15 years. The graduate engineer appears to have an advantage from that time on to about 45 years, when the curves again come together.

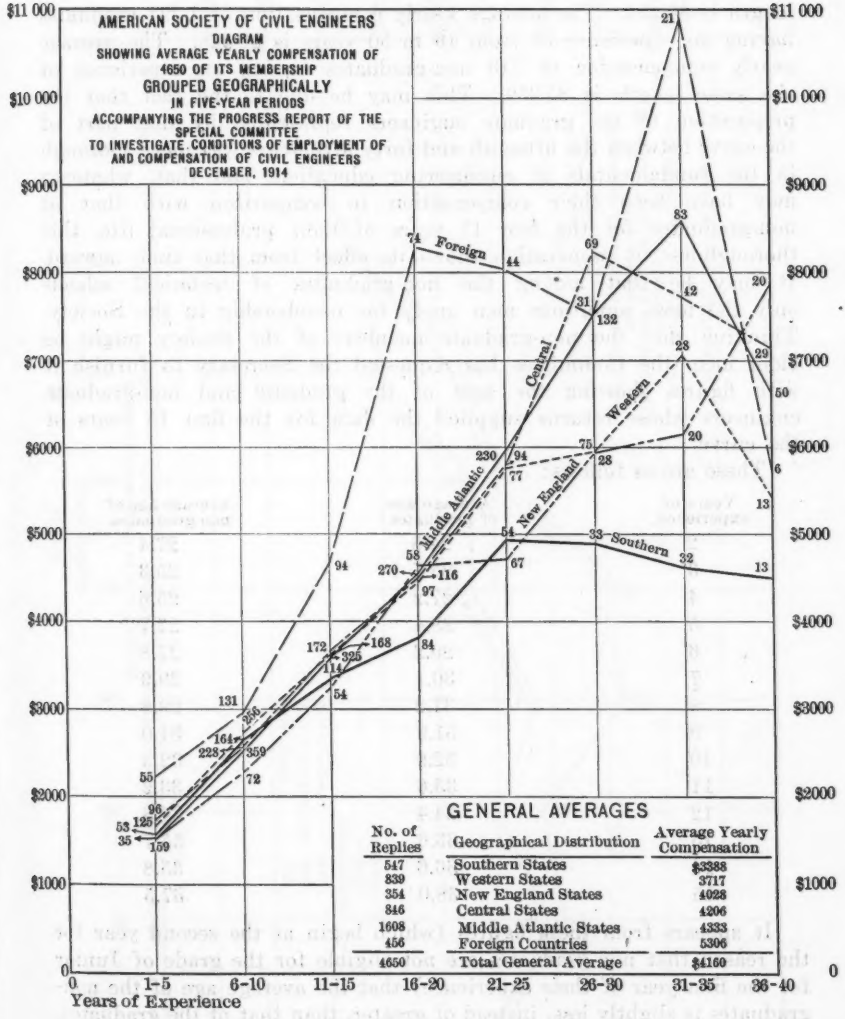

AMERICAN SOCIETY OF CIVIL ENGINEERS
DIAGRAM
SHOWING AVERAGE YEARLY COMPENSATION OF
4650 OF ITS MEMBERSHIP
GROUPED GEOGRAPHICALLY
IN FIVE-YEAR PERIODS
ACCOMPANYING THE PROGRESS REPORT OF THE
SPECIAL COMMITTEE
TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF
AND COMPENSATION OF CIVIL ENGINEERS
DECEMBER, 1914
$11 000
$10 000
$9000
$8000
$7000
$6000
$5000
$4000
$3000
$2000
$1000
0
Foreign
Central
Western
Middle Atlantic
New England
Southern
1-5
6-10
11-15
16-20
21-25
26-30
31-35
36-40
Years of Experience
GENERAL AVERAGES
No. of Replies
Geographical Distribution
Average Yearly Compensation
547 Southern States $3388
839 Western States 3717
354 New England States 4028
846 Central States 4206
1608 Middle Atlantic States 4333
456 Foreign Countries 5306
4650 Total General Average $4160

FIG. 2.

The average yearly compensation of 1 998 graduates having an experience of from 2 to 15 years is $2 756. The average yearly compensation of 614 non-graduates having an experience of the same length is $3 154. The average yearly compensation of 1 394 graduates having an experience of from 16 to 50 years is $6 328. The average yearly compensation of 719 non-graduates having an experience of the same length is $5 359. This may be due to the fact that the preparation of the graduate engineers represented by that part of the curve between the fifteenth and forty-fifth years was more thorough in the fundamentals of engineering education, and that, whatever may have been their compensation in comparison with that of non-graduates for the first 15 years of their professional life, this thoroughness of preparation shows its effect from that time onward. It may be that among the non-graduates of technical schools only the most ambitious men apply for membership in the Society. Thinking that the non-graduate members of the Society might be older men, the Committee has requested the Secretary to furnish it with figures showing the ages of the graduate and non-graduate engineers whose returns supplied the data for the first 15 years of the curve.

These are as follows:

Years of experience.	Average age of graduates.	Average age of non-graduates.
2	24.8	27.4
3	25.7	25.3
4	27.2	25.6
5	28.4	27.7
6	29.2	27.8
7	30.1	29.9
8	31.2	29.8
9	31.9	31.0
10	32.9	32.3
11	33.6	33.2
12	34.4	34.8
13	35.6	35.7
14	36.6	35.8
15	38.0	37.5

It appears from these figures (which begin at the second year for the reason that non-graduates are not eligible for the grade of Junior for the first year of their experience) that the average age of the non-graduates is slightly less, instead of greater, than that of the graduates.

The replies from engineers having more than 50 years of experience are so few, and the information given by them is of so little significance, that it has not been included in any of the diagrams except that giving all the returns without classification.

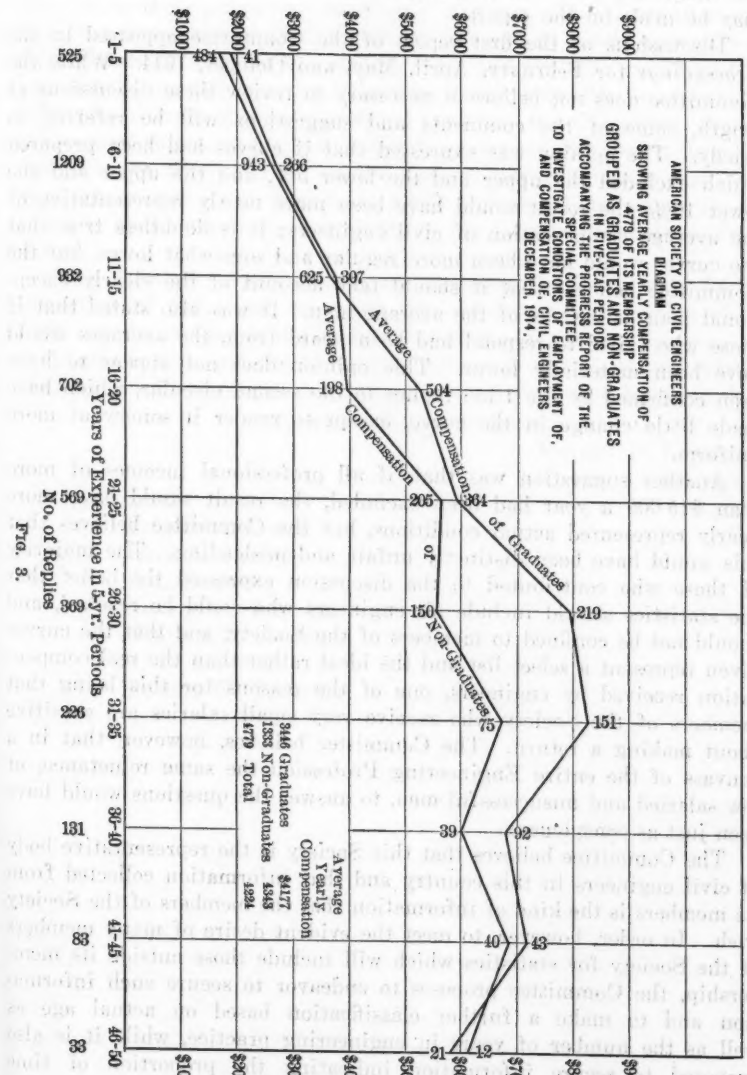

AMERICAN SOCIETY OF CIVIL ENGINEERS
DIAGRAM
SHOWING AVERAGE YEARLY COMPENSATION OF
4779 OF ITS MEMBERSHIP
GROUPED AS GRADUATES AND NON-GRADUATES
IN FIVE-YEAR PERIODS
ACCOMPANYING THE PROGRESS REPORT OF THE
SPECIAL COMMITTEE
TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF,
AND COMPENSATION OF, CIVIL ENGINEERS
DECEMBER, 1914.
Average Compensation of Graduates
Average Compensation of Non-Graduates
Years of Experience in 5-yr. Periods
No. of Replies
Average Yearly Compensation
3446 Graduates $4177
1333 Non-Graduates 4343
4779 Total 4224
1-5
6-10
11-15
16-20
21-25
26-30
31-35
36-40
41-45
46-50
525
1209
962
702
569
369
226
131
83
33
484
41
943
296
625
307
198
504
205
364
150
219
75
151
39
92
40
43
21
12
$1000
$2000
$3000
$4000
$5000
$6000
$7000
$8000
$9000

FIG. 3.

Other conclusions may be drawn from the information shown by the diagrams, and will probably be brought out in comments which may be made on the report.

Discussions of the first report of the Committee appeared in the *Proceedings* for February, April, May, and October, 1914. While the Committee does not believe it necessary to review these discussions at length, some of the comments and suggestions will be referred to briefly. The opinion was expressed that if curves had been prepared which excluded the upper and the lower 5%, and the upper and the lower 10%, the result would have been more nearly representative of the average compensation of civil engineers; it is doubtless true that the curves would have been more regular and somewhat lower, but the Committee believes that it should take account of the clearly exceptional man as well as of the average man. It was also stated that if those who failed to respond had been heard from, the averages would have been materially lower. This opinion does not appear to have been confirmed by the 1 158 replies to the second circular, which have made little change in the curve, except to render it somewhat more uniform.

Another suggestion was that, if all professional incomes of more than $15 000 a year had been excluded, the result would have more nearly represented actual conditions, but the Committee believes that this would have been distinctly unfair and misleading. The majority of those who contributed to the discussion expressed the belief that the statistics should include all engineers who could be reached, and should not be confined to members of the Society, and that the curves given represent a select list and the ideal rather than the real compensation received by engineers, one of the reasons for this being that members of the Society who receive very small salaries are sensitive about making a return. The Committee believes, however, that in a canvass of the entire Engineering Profession the same reluctance, of low salaried and unsuccessful men, to answer the questions would have been just as conspicuous.

The Committee believes that this Society is the representative body of civil engineers in this country and that information collected from its members is the kind of information that the members of the Society wish. In order, however, to meet the evident desire of many members of the Society for statistics which will include those outside its membership, the Committee proposes to endeavor to secure such information and to make a further classification based on actual age as well as the number of years in engineering practice, while it is also proposed to secure information indicating the proportion of time when engineers are out of employment. In securing information concerning engineers who are not members of the Society, the Committee will be obliged to use such data as it can secure from railroads,

municipalities, and other organizations or individuals employing engineers, and from technical schools which have compiled statistics concerning their graduates and former students. The Committee realizes that the collection of this kind of data will be much more difficult than the work which it has already done and that the results will be less conclusive and less accurate than those herewith presented. If it is the pleasure of the Society to continue the Committee for another year, this work will be undertaken, and, in this event, the Committee hopes that members of the Society will voluntarily send to it any information they may have which will be of assistance in this work, or that such data be presented in the form of discussions of this report.

The Secretary of the Society has prepared the forms used for the collection of data, and has, with the assistance of members of his staff, tabulated the results and prepared the diagrams which accompany this report, and the Committee wishes to record its appreciation of the careful and thorough manner in which this work has been done.

Respectfully submitted,

NELSON P. LEWIS,
Chairman.

COMMITTEE:

NELSON P. LEWIS,
JOHN A. BENSEL,
S. L. F. DEYO,
DUGALD C. JACKSON,
WILLIAM V. JUDSON,
C. F. LOWETH,
GEORGE W. TILLSON.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

By Messrs. A. C. Janni and William Cain.

Mr. Janni.

A. C. Janni, M. Am. Soc. C. E. (by letter).—In a recent paper by William Cain, M. Am. Soc. C. E., entitled "Stresses in Wedge-Shaped Reinforced Concrete Beams", there were enunciated formulas for computing the maximum compressive stresses in concrete, as well as the maximum tension in steel. These formulas, together with the assumptions underlying them, are of such a nature as to lend themselves to a critical study which, the writer thinks, may not be entirely devoid of interest.

Leaving out of consideration the points of minor importance in this paper, it is thought expedient to confine this critical analysis to the principal ones. In the first place, in dealing with the problem referred to, the author, in simplifying a certain expression, uses a method somewhat objectionable.

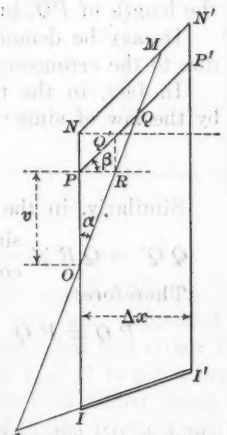


Fig. 16.

Fig. 16 is similar to the author's Fig. 2 (*b*), and shows two parallel sections, NI and $N'I'$, of a wedge-shaped reinforced concrete beam. Let O be the position of the neutral axis of the section, NI, and MJ the position of NI after stress; the angle of rotation being α.

A foot-note on page 748 of the paper is as follows: "In the triangle, POQ, by the law of sines, $PQ = \frac{v \sin. \alpha}{\cos. (\alpha + \beta)}$. As α tends toward zero, this approaches $\frac{v\,\alpha}{\cos. \beta} = v\,\alpha \cos. \beta$."

* This is a continuation of the discussion on the paper by William Cain, M. Am. Soc. C. E., published in *Transactions*, Vol. LXXVII, p. 745.

Mr. Janni.

This simplification is not correct. A mathematical simplification cannot be performed on a mathematical abbreviation. Now, cos. $(\alpha + \beta)$ is purely a symbolical expression, and one cannot operate on it without developing first the expression itself.

Cos. $(\alpha + \beta)$ is not an actual value, and can be an actual value only when α and β have assigned numerical values, which is not so in this case.

The correct simplification of this expression is represented by:

$$P\,Q = \frac{v\,\alpha}{\text{cos. } \beta - \alpha \text{ sin. } \beta}$$

which cannot be simplified further, when one considers that $v\ \alpha$ and α sin. β are infinitesimals of the same order, and that the previous expression is not final but must pass through an integration, as is shown in the paper.

It may be readily seen that the author, with that simplification, did not find the value of PQ, as he thinks, but of a part of it only.

In fact, in the triangle, POR, Fig. 16, α being very small, $PR = \alpha\ v$; and RQ' being parallel to OP, or perpendicular to PR, $PQ' = \frac{\alpha\ v}{\text{cos. } \beta} = \alpha\ v \text{ sec. } \beta$; therefore, the value $\alpha\ v$ sec. β does not represent the length of PQ, but that of PQ', which is a part of PQ.

It may be demonstrated also that the difference, QQ', is precisely due to the erroneous simplication.

In fact, in the triangle, PRQ, with the notations in Fig. 16 and by the law of sines:

$$R\,Q = \frac{v\ \alpha \text{ sin. } \beta}{\text{cos. } (\alpha + \beta)}.$$

Similarly, in the triangle, RQQ':

$$Q\,Q' = Q\,R \times \frac{\text{sin. } \alpha}{\text{cos. } \beta} = \frac{\text{sin } \alpha\ \ v\ \alpha\ \text{ sin. } \beta}{\text{cos. } \beta\ \text{ cos. } (\alpha + \beta)} = \frac{\alpha^2\ v}{\text{cos. } (\alpha + \beta)} \text{ tan. } \beta.$$

Therefore:

$$P\,Q = P\,Q' + Q'\,Q = v\ \alpha \text{ sec. } \beta + \frac{\alpha^2\ v \text{ tan. } \beta}{\text{cos. } \beta - \text{sin. } \beta\ \alpha}$$

$$= \frac{v\ \alpha}{\text{cos. } \beta - \text{sin. } \beta\ \alpha}$$

which is the value of PQ, as previously stated by the writer.

It seems to the writer that in the paper referred to, as well as in its closure, there is a confusion of conception in regard to the behavior of a toe-beam, a dam, and counterfort.

A dam, when properly built, may be designed,* as far as the maximum compressive stress is concerned, considering a vertical element of it 1 ft. thick and assuming that, for a certain height, it is a vertical

*F. Platzmann, "Ueber den Querschnitt der Staumauern." Leipzig, 1908.

beam, fixed at the base and free at the top, under the actions of its dead load and of the hydraulic pressure against its up-stream face. Mr. Janni.

The writer does not think it necessary to enter into any analysis concerning the design of a dam; this important kind of construction now has a rather complete theory, which does not need any workable formula for its design.

A toe-beam, supporting a retaining wall, etc., may be designed as a horizontal beam, if its shape is as in Fig. 1,* fixed at one end and free at the other, under the actions of the vertical reaction of the soil and of the friction developed along the base line.

Concerning now the case of a counterfort, as shown in Fig. 17, the writer does not understand on what assumptions it is designed as a beam.

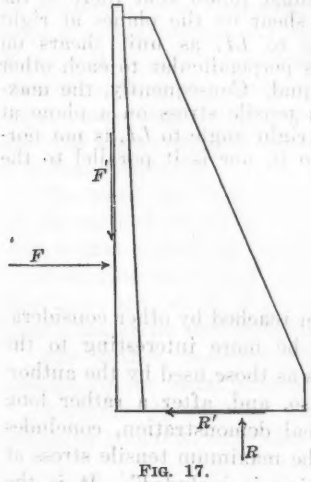


FIG. 17.

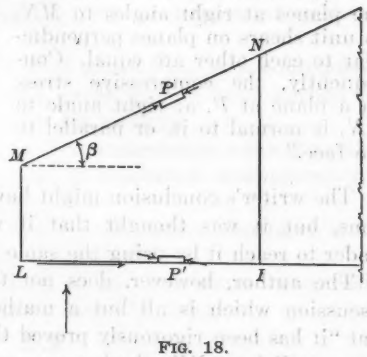


FIG. 18.

If a beam, as commonly understood, means a solid having one dimension a great deal larger than the other two, and supported or fixed at both ends, or fixed at one end, having the other end either free or supported, it is clear that the solid shown in Fig. 17 is not a beam.

That solid is under the actions of the horizontal thrust along its height, as well as of the reaction at the base and of the friction against the soil; there is no moment at the base which would entail the fixed-end hypothesis.

The author is quite right in demonstrating that the maximum compressive stress along *MN*, Fig. 18, is parallel to *MN*, and the writer begs leave to borrow that demonstration, in order to show that in the same beam the maximum tensile stress is not parallel to *LI*, if friction is included.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 745.

Mr. Janni.

For convenience the writer will quote the author's demonstration, and alongside of it will place the demonstration supporting his own statement:

Author's demonstration that the direction of the stresses at any point of the upper surface is parallel to *MN*.

" * * *; for, consider a small rectangular parallelopiped of the concrete at *P*, with faces parallel and perpendicular to *MN*. There can be no shearing stress on *MN*, as no part of the beam extends above *MN* to produce shear, and there is no external force acting on *MN* except the atmospheric pressure, which does not exert any friction on the face. It must follow that there can be no shear on the planes at right angles to *MN*, as unit shears on planes perpendicular to each other are equal. Consequently, the compressive stress on a plane at *P*, at right angle to *MN*, is normal to it, or parallel to the face."

Writer's demonstration in support of his statement that the maximum tensile stress at any point of *LI*, is not parallel to *LI*.

In fact, consider a small rectangular parallelopiped of the concrete at *P'*, with faces parallel and perpendicular to *LI*. There is shearing on *LI* developed by the friction due to the soil reaction.

It must follow that there is the same shear on the planes at right angles to *LI*, as unit shears on planes perpendicular to each other are equal. Consequently, the maximum tensile stress on a plane at *P'* at right angle to *LI*, is not normal to it, nor is it parallel to the face.

The writer's conclusion might have been reached by other considerations, but it was thought that it would be more interesting to the reader to reach it by using the same truths as those used by the author.

The author, however, does not think so, and, after a rather long discussion which is all but a mathematical demonstration, concludes that "it has been rigorously proved that 'the maximum tensile stress at *I* is parallel to *NI*' whether or not friction is included". It is the writer's opinion that this demonstration does not demonstrate anything as to the direction of the tensile stress along *LI*.

It is difficult to understand the purpose of the demonstration concerning the well-known continuity of stresses in a beam, brought forth so elegantly by Culmann by means of his "circular diagrams."

When it is assumed that all compressive stresses, with respect to the section, *NI*, are parallel to *MN*, it is going too far.

From the description in the paper of the behavior of those stresses along *NI*, the writer has the impression that the author, perhaps, refers to "principal" or "ideal stresses," instead of bending stresses, as usually called.

If the writer's assumption is correct, then it cannot be assumed that they are parallel, neither can it be assumed that, with respect to the

Mr. Janni.

section, *NI*, plane sections before stress remain plane sections after stress.

Theory clearly shows the behavior of those stresses and the shape of *NI* after stress.

Fig. 9* shows the lines of the ideal or principal stresses in the toe, *MNIL*, under the action of the vertical reaction. This figure may be obtained, for instance, by cutting Fig. 13† at 4-4 or 5-5.

Obviously, those stresses, with respect to *NI*, are not altogether compressive stresses, but, at each point of *NI*, the component of it, normal to *NI*, will be the compressive stress, and the component of it, parallel to *NI*, will be the shear at that point; this being quite an elementary matter.

Speaking of compressive stresses not being normal to a cross-section is nonsense. Furthermore, the stretching of the fiber, *PP'*, with respect to the section, *NI*, is a function of the modulus of elasticity, *E*, as well as of the coefficient of Poisson, and not of the modulus of elasticity, *E*, only. However, whatever may be the meaning of the paper concerning the nature of those internal forces, the writer thinks that the author entered a rather difficult field when he tried to devise a workable formula.

The mere fact that the shear at *N* is never zero, as a consequence of the method of design shown in the paper, should have warned the author that something was wrong with his method.

Following his method, the shear at *N* is a maximum when $\beta = 45°$; that is to say, at *N*, if $\beta = 45°$, there is maximum compression and maximum shear simultaneously.

The author, however, states: "Above the neutral surface, it decreases, by the usual parabolic law, to zero at *N*."

Formula 8‡ for the computation of the maximum compressive stress in concrete, may also be written (see Fig. 10§):

$$f_c = \frac{2M}{by\left(h - \frac{1}{3}y\right)\cos.^2\beta}$$

where: f_c = maximum compressive stress in concrete,
M = external moment,
β = angle of inclination of the top of the beam to the horizontal,

the other letters having the meaning shown in Fig. 10.

This formula shows that, if *N* keeps a constant section, and *MN* rotates about *N*, the angle, β, increasing gradually, while *M* keeps constant also, the *NI* becomes weaker and weaker.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 765.
† *Loc. cit.*, p. 771.
‡ *Loc. cit.*, p. 751.
§ *Loc. cit.*, p. 758.

Mr. Janni.

Neither does the limitation of β to 60° for the permissible use of the author's formula prevent the foregoing absurd conclusion; nor should his remark, that the line, *MN*, should be extended to the left of the section, *NI*, be taken into serious consideration.

It is well known that the left part of that solid remains there only and solely in order to carry the action of the external force, which is acting on it, on the section, *NI*; provided the acting external force has constant intensity, direction, and distance from the section, *NI*. Its moment (external moment) will not change in the least its value, whatever be the geometrical shape of the solid on the left of *NI*.

If the author, in dealing with wedge-shaped beams, as in Fig. 18, had considered the cross-section at *N* normal to *MN*, instead of parallel to the acting forces, he would have avoided absurdities, and would have seen that, after all, theory is not "plainly inadequate".

The author says:

"Suppose a reinforced concrete T-beam, Fig. 15, with an inclined flange; would not a designer naturally take the compressive stresses in the flange as acting parallel to its surface?"

To this the writer would like to reply that a designer would take the compressive stresses, as the author says; but he would not consider those compressive stresses as acting on the vertical section, as shown in Fig. 15; if he did, he would be flatly wrong.

The case of a toe-beam, as in Fig. 18, is not one of bending only, but of bending and compression at the same time.

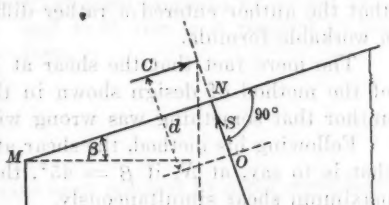

FIG. 19.

Fig. 19 shows how the vertical reaction, *R*, acts on a toe-beam. In this figure:

C = Component normal to NI = the intensity of compressive force acting on NI;

$C \times d$ = External moment of the compressive force, C, acting on NI;

S = Component parallel to NI = shear along NI.

It would be rather interesting to know what are the "mathematical difficulties in the way where" a section "is not parallel to the line of action of the loads."

The writer has reason to believe that, in the location of the neutral axis in a cross-section of a wedge-shaped beam, the elementary principles, which are the basis of such determination, have been ignored.

In fact, assuming that all stresses have the same direction and

that plane sections before stress will remain plane sections after stress, is the same as assuming that the stresses are proportional to the distances from the neutral axis; no matter what the common direction of those stresses may be. Mr. Janni.

In other words, the center of compression, being fixed in position, as well as the center of tension, the position of the neutral axis is fixed also, no matter what may be the intensity of the stresses (within the elastic limits) and their direction.

The center of compression, after all, is the antipole of the neutral axis with respect to the ellipse of inertia of the compressed part of the section, and the center of tension is the antipole of the neutral axis with respect to the ellipse of inertia of the part of the section under tensile stress.

This relation is purely a graphical one, and as long as the centers of compression and tension do not change their position, that of the neutral axis cannot change.

The statement, therefore, that the plain methods used to locate the neutral axis in a cross-section of a prismatic beam "are entirely inapplicable to a wedge-shaped beam" is quite incorrect. This remark, however, is purely incidental; because, in the case under discussion, the position of the neutral axis is determined either by an equation of the third degree, or by graphical methods.

Equation 3 (a quadratic in k), obtained by the author, should have warned him again that his theory was astray; for, a quadratic equation for determining the position of the neutral axis is obtained when it is a case of bending only; but, in the case of a reinforced concrete toe-beam, as in Fig. 18, as has been said already, there is both bending and compression at the same time.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The writer is again indebted to Mr. Janni for a stimulating discussion in which he introduces some new points of view. Since the publication of his paper,† the writer has attacked the subject from another point of view and instituted a comparison of methods which leads to some results of interest. He intended to publish this independently, but it can also appropriately be given here in connection with Mr. Janni's further discussion. Mr. Cain.

Mr. Janni devotes much space to the expression, $P\ Q = \frac{v \sin. \alpha}{\cos. (\alpha + \beta)}$. According to the writer, as α tends indefinitely toward zero, this expression tends toward the value, $\frac{v\ \alpha}{\cos. \beta} = v\ \alpha \sec. \beta$. The writer is unable to catch Mr. Janni's viewpoint in objecting to this statement,

* Chapel Hill, N. C.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 745.

Mr. Cain. Mr. Janni contending that cos. $(\beta + \alpha)$ does not tend to cos. β when α tends to zero. A number of proofs will be given, in the hope that one, at least, may meet his objections.

(1) From the unit circle, Fig. 20, in which, $P\ C\ M = \beta$, $M\ C\ N = \alpha$, cos. $(\beta + \alpha) = C\ A$, cos. $\beta = C\ B$, we see that as α tends toward zero, N approaches M, A approaches B, and $C\ A$ approaches $C\ B$ indefinitely, which proves that cos. $(\beta + \alpha)$ approaches cos. β indefinitely as α tends toward zero. Further, cos. $(\beta + \alpha) = C\ A$, is as much a "value" as cos. $\beta = C\ B$.

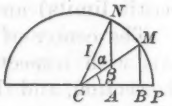

FIG. 20.

(2) Cos. $(\beta + \alpha)$ = cos. β cos. α — sin. β sin. α. As α tends toward zero, the right member tends toward cos. β — 0 = cos. β. Otherwise, sin. β sin. α is an infinitesimal and can be neglected in comparison with the finite first term cos. β cos. α, which by the same theory of infinitesimals can be replaced by cos. β.

(3) From Fig. 21, $R\ Q = O\ R$ tan. α, $P\ Q = R\ Q$ sec. $\beta = O\ R$ tan. α sec. β; therefore, as α approaches zero, Q approaches P, $O\ R$ approaches $O\ P$, and tan. α approaches α; whence $P\ Q$ approaches indefinitely the value,

$$O\ P.\ \alpha \text{ sec. } \beta = v\ \alpha \text{ sec. } \beta.$$

(4) Finally, take the expression deduced by Mr. Janni:

$$P\ Q = v\ \alpha \text{ sec. } \beta + \frac{\alpha^2\ v \text{ tan. } \beta}{\text{cos. } \beta - \text{sin. } \beta\ .\ \alpha}.$$

The last term, containing α^2 as a factor, is an infinitesimal of the second order in comparison with the preceding term having α as a factor; hence, by a well-known theory of infinitesimals, the last term can be omitted in comparison with the preceding, giving, as before, $P\ Q = v\ \alpha$ sec. β.

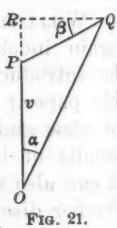

FIG. 21.

Turning now to the subject of principal stresses, the term "principal stress" in the case of beams subject to flexure, is to be understood in the usual sense as defined in the textbooks. Thus, Morley,* in addition to the usual figure, showing the directions of the principal stresses at various points in a short prismatic beam of rectangular cross-section, gives the magnitudes of the principal stresses for all points in a vertical section where the shear is large. A figure corresponding to this, but to a larger scale, is given by Turneaure and Maurer.†

In the case of the wedge-shaped beam without reinforcement, Mr. Janni's Fig. 9‡ represents the directions of the principal stresses at

* "Strength of Materials," p. 138.
† "Principles of Reinforced Concrete Construction," Art. 46.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 765.

the various points of the cross-section approximately, about as the writer would conceive them. It was because Mr. Janni identified these intermediate "principal stresses" with the writer's "intermediate bending stresses" that he objected. In the hope that the distinction may be defined clearly, an attempt has been made to draw the principal compressive stresses at various points on the section, *I N*, of Fig. 22 (3), from analogy to the figures of the textbooks referred to. The principal tensile stresses on *O N* are not drawn. The figure is supposed to represent part of the toe of a reinforced concrete beam, subjected to a load acting vertically upward. The upper part of the beam is supposed to make an angle, β, with the horizontal. Mr. Cain.

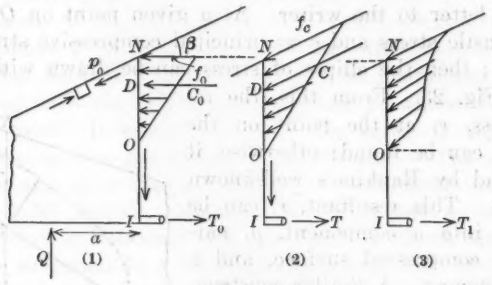

FIG. 22.

Professor O. H. Basquin,* has kindly sent the writer what he styles a "Rough Test of a Wedge-Shaped Glass Beam", with polariscope, to show the inclination of the principal compressive stresses with the horizontal at the points indicated in Table 2. The value of β was 15.5° and the small beam was subjected to a single vertical load about 3½ in. from the section where the measurements were taken. The entire vertical section was divided into fourteen equal parts, and the center of each (where the angle was measured) was numbered from the top down, consecutively, 1, 2, 3, ... The results are given in Table 2.

TABLE 2.

Point.	Angle.	Point.	Angle.	Point.	Angle.	Point.	Angle.
1	15.6°	5	17.7°	9	76.4°	13	89.2°
2	14.9	6	22.2	10	83.9	14	89.5
3	14.4	7	35.9	11	86.7		
4	15.9	8	60.5	12	88.0		

"The values given for angles away from center of beam may have an error of about half a degree with a larger probable error near the center."

* Northwestern University, Evanston, Ill.

Mr. Cain.

The results are very interesting. It will be observed that the angle first decreases slightly and then increases. For one-fourth of the way down, it is practically constant and nearly equal to β.

The writer's assumption as to his so-called "bending stresses" is shown in Fig. 22 (2). These stresses, for a given β, are all taken parallel to the upper surface, and increase regularly in going from the neutral axis, O, to N.

If, in Fig. 22 (3), the tensile principal stresses acting on $O\ N$ are supposed to be added, the relation between the state of stress in the revised figure (which represents all the stresses acting on the section, $I\ N$) and that given in Fig. 22 (2) can be shown by aid of the ellipse of stress in a manner kindly suggested by Professor H. A. Thomas,* in a recent letter to the writer. At a given point on $O\ N$, let $t =$ principal tensile stress and $c =$ principal compressive stress (at right angles to t); then the ellipse of stress can be drawn with t and c as semi-axes, Fig. 23. From this, the resultant stress, r, at the point on the plane, $I\ N$, can be found; otherwise, it can be found by Rankine's well-known construction. This resultant, r, can be decomposed into a component, p, parallel to the compressed surface, and a vertical component. A similar construction being supposed to be effected for each point of $O\ N$, the fundamental assumption made by the writer can be stated thus: the component of the resultant stress, r, parallel to the surface in compression, at any point of the reference plane, $I\ N$, is assumed to increase directly as the distance of the point from the neutral axis. The neutral axis may be defined as the line where the parallel component vanishes.

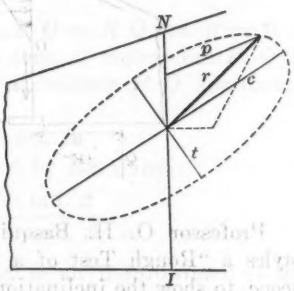

FIG. 23.

This is in agreement with the common theory of flexure for prismatic beams, which states that "the normal component of the resultant stress on the reference section, at any point, is assumed to increase directly as the distance of the point from the neutral axis."

This use by Professor Thomas of the ellipse of stress furnishes a scientific basis for the system of parallel stresses proposed, and shows very clearly that no one of the parallel stresses is a principal compressive stress, except at N, where the principal stress is parallel to the surface. Further, when this construction is supposed to be made for each point of $O\ N$, Fig. 22 (3), the result will be, not only the system of parallel forces, Fig. 22 (2), but, in addition, a vertical force acting downward along $N\ O$, equal to the sum of the vertical

* Rose Polytechnic Institute.

components referred to in connection with Fig. 23. This force, together with the sum of the vertical components of the parallel stresses, constitutes the total vertical shear on the part of the section under compression. As just shown, the parallel stress at some point, O, is zero, and if the hypothesis that plane sections, I N, remain such after strain is realized, then the original analysis shows (at least for a constant, E), that the linear law of variation holds, or the parallel stresses increase uniformly from O to N. Mr. Cain.

It thus appears that the system consisting of the uniformly varying parallel stresses of Fig. 22 (2), with the vertical force acting along N O, is the exact equivalent of the stresses shown in Fig. 22 (3), with the tensile principal stresses added. It follows that the moment of the system of Fig. 22 (2) about I (say), is exactly equal to the moment about I of the stresses of the other system. This tacitly assumes that the point, O, of Fig. 22 (2), as determined by the formula, is identical with the point, O, of System 3, where the parallel component is zero. It would seem that any error, in this particular, is mainly due to the hypotheses being inexact. The conservation of plane sections after strain, may not be sufficiently realized, particularly near N; besides, the modulus, E, is not a constant when β is not zero; for Poisson's ratio is really involved, since the interior fibers are subjected to stresses at right angles to their axes in addition to the longitudinal stresses. But Poisson's ratio is said to be very small for concrete $\left(\frac{1}{10}\text{ to }\frac{1}{8}\right)$, and the error resulting from ignoring it should be small. Of course, in place of the last two hypotheses, the formulas could all be derived from the one assumption of uniformly increasing stresses, but perhaps the results would not be so obvious.

It may be well to remark that p, in Fig. 23, is an oblique stress, per unit of area of the vertical plane, whereas the unit stress, f, of Fig. 22 (2) and of the formulas, acts normally to a plane perpendicular to the surface; therefore, $p \times 1 = f \cos.\ \beta$, or $f = p \text{ sec. } \beta$. Hence f varies with p and thus follows the linear law with p.

The writer will now try to make clear the term "intermediate bending stress" in connection, say, with a homogeneous toe-beam. In Fig. 23, if r is resolved into a vertical component and a component, p', the inclination of which to the horizontal at N is β, and which is then decreased according to some arbitrary law, gradually from β at N, to zero at I, as the point considered moves down the joint, the system of stresses, p', would constitute the system of "intermediate bending stresses." Below O (where $p' = 0$) the stresses would be tensile. Such a conception of "intermediate bending stresses" was entertained to try to bridge over the gap between the states of stress of reinforced and homogeneous toe-beams; it was not used, and is not necessary, in the development of the formulas, and it will be

Mr. Cain.

entirely eliminated from the discussion that follows. In this, a discussion of such experiments as are at present available will be given the better to estimate the degree of security of the formulas.

From the experiments on dams, discussed later, it seems highly probable that plane sections, $I\ N$, do not remain such after strain in the vicinity of N. In fact, the resistance there, to the left of $I\ N$, is less than for a prismatic beam of the height, $I\ N$. If so, the value of f_c, Fig. 22 (2), as computed from the formula, will be in excess, or the result will be on the side of safety, as originally claimed. Direct experiments on wedge-shaped beams could easily be made to decide as to whether the conservation of plane sections after strain is sufficiently realized; also to determine the neutral axis and the values of f_c and other parallel stresses near the surface. It is the writer's belief that some of the latter are nearly as large as the computed f_c and that the actual principal compressive stress at N is less than f_c as computed. In fact, if the latter statement is true, then certain stresses below N must be largely increased, so that the resisting moment about I may equal the constant moment of the external forces. As a comparison will ultimately be made between the results pertaining to the systems of stress shown in Fig. 22 (1) and (2), the first system (1) will now be examined. In this, the direct stress at N, f_0, due to bending (using Morley's term) acts horizontally, and is to be found by the well-known formula for a prismatic beam, just as if β was zero.

$$f_0 = \frac{2\,M_c}{k_0\,j_0\,b\,d^2}.$$

This formula is found in any of the textbooks on reinforced concrete, or it is given by Equation 12 of the writer's paper,* provided k_0 and j_0 denote the values of k and j for $\beta = 0$, the steel bars being horizontal. Since the moment of the external forces about I is exactly equal to the moment of the internal stresses about I, we can put $M = M_c$. For brevity, write,

$$\frac{2\,M_c}{b\,d^2} = \frac{2\,M}{b\,d^2} = c\,;$$

then the formula can be written,

$$f_0 = \frac{c}{k_0\,j_0}.$$

It was long ago pointed out, in the case of dams at least, that f_0 was only one conveniently chosen component of the resultant stress at N, which last acts parallel to the surface on a plane perpendicular to it. Call this principal stress p_0; then if f_0 is really the true

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 754.

normal unit stress on a vertical plane, it follows that $p_0 = f_0 \sec.^2 \beta$.* Mr. Cain.

Now, for large values of β, with M and therefore f_0 constant, it is seen that $p_0 = f_0 \sec.^2 \beta$ becomes very large, increasing indefinitely toward infinity as β approaches 90 degrees. As this is totally unreasonable, it must follow, for a given M, b, and d, that f_0 must decrease as β increases, so that $p_0 = f_0 \sec.^2 \beta$, shall remain within reasonable limits as β increases. Also since $\sec.^2 \beta$ is a continuous function when $\beta < 90°$, increasing with β, and since $p_0 = f_0$, for $\beta = 0$, is an accepted value, it seems very probable that f_0 decreases continuously as β increases from zero, the decrease being small for small values of β and becoming greater and greater as larger values of β are taken. This probability becomes a certainty when it is considered that the toe to the left of $I\ N$, Fig. 22 (1), is less resisting than a prismatic beam of the cross-section, $I\ N$, of the same length, and consequently the actual value of f_0 at N, for the toe, is less than that for the prismatic beam of the same length. This is easily realized, especially for large values of β, by supposing the toe to be cut out of the corresponding prismatic beam, when it is apparent that near N, there is less matter to resist horizontal forces to the left than to the right of $I\ N$, and the conservation of plane sections after strain, may not apply there.

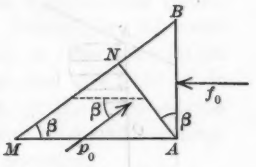

FIG. 24.

It appears, further, that the excess in the computed value of f_0, over the true value for the toe, should be very small near the truncated end of the toe, since the resistance, as regards bending of the toe and the corresponding prismatic beam (the common section, $I\ N$, being now near the end of the toe) are nearly equal. As the section, $I\ N$, Fig. 22, is taken farther from the end of the toe, the excess should increase and reach a maximum where the toe joins the stem.

The state of the horizontal stress on the vertical plane, $I\ N$, is probably as indicated in Fig. 25, the unit stress at N being less than the value of f_0 as computed by the foregoing formula, the unit stress then increasing for a little distance below N and then decreasing as O is approached, the sum of the moments of all the stresses on the cross-section about I being equal to M, as before. The unit stresses, a little below N, are thus greater than the ordinary theory would give, and plane sections, as $I\ N$, do not remain so after bending, otherwise the stresses would follow a linear law of variation from

* In Fig. 24, let $A\ N$ be at right angles to the surface, $M\ N$. Then, when $A\ N$ is regarded as infinitesimal, the stress on it is, $p_0 . A\ N = p_0 . A\ B \cos. \beta$, and its horizontal component is, $p_0 . A\ B \cos.^2 \beta$. For equilibrium of $A\ N\ B$, this must equal the horizontal stress on $A\ B = f_0 . A\ B$. On equating and solving, we find, $p_0 = \frac{f_0}{\cos.^2 \beta} = f_0 \sec.^2 \beta$, a formula that seems to be better known in Europe than in America.

Mr. Cain.

O to N. The same reasoning would indicate a similar modification in the state of stress shown in Fig. 22 (2), so that f_c, as computed by the formula, is in excess of the true value. It is to be hoped that direct experiments on wedge-shaped beams will soon be instituted to test these inferences. Lacking these, the experiments on india-rubber dams of Wilson and Gore and those on plasticine dams of Ottley and Brightmore* will be appealed to. From these experiments it was ascertained that the vertical unit stress on horizontal planes did not vary uniformly, the stresses near the down-stream face being generally less than corresponds to the law of uniform variation, the difference being practically nothing in the upper part of the dam and increasing as the foundation is approached, where it is comparatively large.

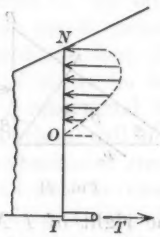

FIG. 25.

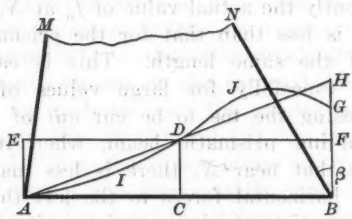

FIG. 26.

The variation of this stress is shown in Fig. 26, which may be taken as typical of the results of the various experiments. In this figure, $A\ B$ is a horizontal section of the dam, $A\ B\ N\ M$, near its base, the angle, β, being 40° 55′ for the plasticine dam and 32° for the straight portion of the rubber dam. The resultant on the section was supposed to cut it, $\frac{1}{3}\ A\ B$ from B, so that the vertical ordinates from AB to a straight line, $A\ H$, represent the stresses in question, according to the usual theory. Also, if C is the mid-point of $A\ B$, the vertical ordinates from $A\ B$ to $E\ F$ represent the uniform compression due to the supposed vertical load at C. The ordinates from $D\ F$ to $D\ H$ and from $D\ E$ to $D\ A$ represent the stresses due to the bending moment, all according to the ordinary theory. By experiment, the vertical unit stress on $A\ B$, at any point of it, is given by the vertical ordinate there, from $A\ B$ to the curved line, $A\ I\ D\ J\ G$. The variation of stress due to a central load at C is unknown, but it is certainly not uniform, and the stress is a little less than $B\ F$ at B, and probably a little greater than $C\ D$ at C, the variation being given by the ordinates from $A\ B$ to a slightly curved line between the extreme points, and not by $E\ D\ F$. The actual unit stresses corre-

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–08, Part II, see Pl. 3, Fig. 4, and Pl. 5, Figs. 19–21.

sponding to the bending moment will be represented by the vertical ordinates between this slightly curved line and *A I D J G*. This (for the part, *C B*, of the section) would give about the variation of stress shown in Fig. 25. The unit stress, corresponding to the bending moment, that particularly interests us, is that at *B*, Fig. 26, which is represented by a vertical line from *G* to a point slightly below *F*. This stress will be smaller than *F H*, and, similarly, we should conclude for the toe, Fig. 22 (1), that f_0 as computed is too large (at least for $\beta = 32°$ to $41°$), though the excess over the true value is negligible near the truncated end of the toe and increases as the stem is approached where it is comparatively large. Mr. Cain.

No quantitative results can well be given, but a glance at the diagrams, relating to the experiments on the dams referred to, will show an excess in the total vertical unit stress on a horizontal plane, say one-fifth of the height of the dam above the foundation, of more than 25%, the amount increasing very materially as the foundation is neared. From the relation between this stress and the part of it due to flexure, we must infer a large proportion of the same excess in the computed over the real value of the stress due to direct bending. A similar result then would follow for the toe in question.

From all that precedes, both as to facts and theory, it follows, for the toe, Fig. 22 (1), that f_0, as usually computed, is in excess of the true value, the amount being small for small values of β and increasing with β, slowly at first and more rapidly when β is large, the excess being enormous when β approaches 90 degrees. Further, it would appear that this excess should be very small when the section, *I N*, is taken near the truncated end of the toe, and that it should increase as the vertical section is taken nearer its junction with the stem of the retaining wall, where it is a maximum.

A comparison will now be made between the results obtained in computing the principal stress at *N*, (1) by the formula previously given,

$$p_0 = f_0 \sec.^2 \beta = \frac{c}{k_0 j_0} \sec.^2 \beta$$

and (2) by use of Equation 15 of the writer's paper,*

$$f_c = \frac{2M}{kj\, bd^2 \cos.^2 \beta} = \frac{c}{kj} \sec.^2 \beta,$$

where, $c = \frac{2M}{bd^2}$, Fig. 22 (2), showing the assumed distribution of stresses along the vertical section.

On dividing the first equation by the second, we have,

$$\frac{p_0}{f_c} = \frac{kj}{k_0 j_0}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 756.

Mr. Cain.

where k, j, refer to System (2) of stresses, the values being taken from the full lines of the diagram, Fig. 6, of the writer's paper,* for the assumed value of β. The values of k_0, j_0, may be found on the same diagram from the line corresponding to $\beta = 0$.

The results are given in Table 3 for various values of β and $p = \frac{A}{b\,d}$.

TABLE 3.—STEEL PERCENTAGE, 100 p.

$100\,p =$	0.2	0.6	1.0	2.0
	$\frac{p_0}{f_c}$	$\frac{p_0}{f_c}$	$\frac{p_0}{f_c}$	$\frac{p_0}{f_c}$
$\beta = 10°$	1.02	1.02	1.01	1.00
20°	1.05	1.05	1.04	1.02
30°	1.13	1.11	1.09	1.06
40°	1.24	1.20	1.17	1.12

In Fig. 22, $d = I\,N$, b = constant width of section
In Fig. 22 (1), $k_0\,d = N\,O$, $j_0\,d = D\,I$.
In Fig. 22 (2), $k\,d = N\,O$, $j\,d = D\,I$.

Recalling that p_0 is always in excess of the true value, the excess appreciably increasing with β, the values of f_c, corresponding to System 2 of Fig. 22, are seen to give consistent results and in the right direction. The regularity of the changes in the ratio is noticeable, and the near approach to unity for $\beta = 10$ to 20 degrees. The excess of p_0 over f_c, which varies from 0 to an extreme of 24% for $100\,p = 0.2$, $\beta = 40°$, is what might have been expected from the diagrams relating to the experiments. Also, since the computed value of p_0 for the toe, is in excess (much more than 25%) near or at its junction with the stem for $\beta = 32°$ or $\beta = 41°$, reasoning from the results of the experiments on dams, it is seen from Table 3 that f_c for $\beta = 32°$ or $\beta = 41°$, is in excess of the true value, and, presumably, f_c is in excess for any value of β considered.

It is impossible to draw certain conclusions when experiments in great variety are lacking, but all the results, theoretical and experimental, given thus far tend to support the approximate formula for f_c, as a good practical formula within the limits $\beta = 0$ to $\beta = 45°$, and on the side of safety. Further, the value of f_c, as just computed, is much nearer the truth than the value of $p_0 = f_0 \sec.^2 \beta$.

It has been suggested that perhaps $f_0 \sec. \beta$ would be preferable to use, as a semi-empirical formula, in place of $f_0 \sec.^2 \beta$, which gives results too large. Calling $p_0' = f_0 \sec. \beta$, the ratio $\frac{p_0'}{f_c}$ was made out for different steel percentages and values of β, as given in Table 4.

* *Loc. cit.*, p. 755.

TABLE 4.—STEEL PERCENTAGE, 100 p.

Mr. Cain.

$100\,p =$	0.2	0.6	1.0	2.0
	$\frac{p'_0}{f_c}$	$\frac{p'_0}{f_c}$	$\frac{p'_0}{f_c}$	$\frac{p'_0}{f_c}$
$\beta = 10°$	1.01	1.01	1.00	0.99
20°	0.99	0.99	0.98	0.96
30°	0.98	0.97	0.95	0.92
40°	0.95	0.92	0.90	0.86

As p_0' is generally less than f_c, until more decisive experimental evidence is forthcoming, it seems unwise to use the smaller value.

Although the previous investigation shows that the value, p_0, is always in excess, and that f_c is nearer the true value, the question at once occurs: why not use, for such an uncertain quantity as concrete, the larger value?

By reference to Table 3, it is seen that p_0 exceeds f_c by from 1 to 5% only, for $10° < \beta < 20°$ and by from 6 to 24% for $30° < \beta < 40°$, for the various steel percentages. The average excess is small; besides, the junction of toe and stem is a recognized point of weakness in concrete construction, so that a decided excess value seems desirable. Further, the method of computation of p_0 is simple and more generally understood than the method by which f_c is computed, which introduces the novel hypothesis of parallel stresses. If this method is adopted in the computation of f_0, Fig. 22 (1), the values of k_0 and j_0 can be found from usual diagrams pertaining to prismatic beams, the principal stress in the concrete computed from the formula, $p_0 = f_0 \sec.^2 \beta$, and the stress in the steel (which is usually the determining factor) found as usual. If p_0 is assigned a limit of say, 650 lb. per sq. in., then $f_0 = 650 \times \cos.^2 \beta$; with this value and the maximum unit stress allowed in the steel, the usual French diagram can be consulted. The serious drawback to the use of this method is that f_0 is known to be in excess, and that the state of stress shown in Fig. 22 (1), cannot be justified. We have seen, though, that the state of stress shown in Fig. 22 (2) is on a more satisfactory basis, in that it introduces β and f_c from the start, and that it may be near the true state, any variation being most marked near N. At any rate, it is a good working hypothesis, consistent with the few experiments available, and to be tested by future more direct experiments. The formulas corresponding to the hypotheses have been developed by rigorous methods, and the resulting values of f_c agree fairly well with the results of the experiments quoted. It seems to be common practice (which Mr. Janni endorses,

Mr. Cain.

when "β is not large") to use f_0 as the "maximum compressive stress" in the concrete, in place of either p_0 or f_c. It would appear from the foregoing investigation, that such practice may be considerably on the danger side, and should be condemned—certainly until more decisive results can be reached from future experiments.

The previous discussion deals with the case where $\beta > 0$ and $\beta_1 = 0$. For the case of the heels and counterforts of retaining walls, $\beta = 0$, and, generally, some of the steel bars are not at right angles to the section, $I\ N$. For this case, all the general formulas of the writer's paper hold on putting $\beta = 0$. This is a much simpler case than the former, since the only hypothesis introduced is that plane sections remain such after bending.

In the writer's first draft of his paper, this case was considered by itself, and, following it, the case where $\beta > 0$, which introduced a second hypothesis as to parallel stresses. Both cases were then combined into one to save space in the final writing. It is regretted now that the two cases were not considered separately, as then the merits or demerits of each could have been more easily seen or discussed.

Taking up again Mr. Janni's further comments, it may be observed that his Fig. 17 shows a buttressed wall, which is not the case of the counterforted wall investigated by the writer. In the latter, the earth is to the right of the face wall, the counterfort being buried in it. The force, F, of Fig. 17, thus acts to the left. In practice, too, the friction forces are usually neglected, for simplicity. The base of the counterfort is connected to the foundation slab (which is loaded with earth) by steel rods, vertical and inclined, so that the counterfort can be regarded as a beam fixed at the ends or at its junction with the base slab. It is true that it is a very short beam, so that the effect of shear is very large; still, a bending moment has to be accounted for, and beam action is thus induced. The theory of such beam action enables one to compute the stresses in the rods, as was illustrated in connection with Fig. 3.*

With regard to the toe-beam, Fig. 18, if the friction is included, the "principal stress" at P' is, of course, inclined to the horizontal; but the writer had in mind the "bending stress", as explained in the foregoing, which was taken as acting parallel to $L\ I$, according to the original assumption. Mr. Janni is right in his remark about the modulus, E, not exactly applying, as before remarked. It was considered, however, that in an approximate computation, the Poisson ratio consideration could be ignored, particularly as in concrete this ratio is very small. In all approximate computations, one or more assumptions, not exactly true, are usually made, it being considered that the degree of accuracy in the final computations is the final test, and, from what has preceded (see Table 4) the computations for f_c

* *Loc. cit.*, p. 751.

stand the test for practical accuracy remarkably well. The ordinary modulus could evidently be used near the surface, where the larger stresses with the larger arms are most effective in resisting the bending couple, so that the error should not be large in using a constant modulus throughout. Mr. Cain.

As to shear, let Fig. 27 represent an infinitesimal rhombic prism at the neutral axis, two faces being parallel to the neutral surface, OO', and the other faces being vertical. The stress on the two vertical faces shown is a pure shear, s. As there is generally a vertical component on all planes parallel to OO' to the compressed surface, the possibility of such a component must be recognized at the neutral surface. Then, if we resolve r, the resultant unit stress on $O\,O'$, into a vertical component, t, and one acting parallel to $O\,O'$, it follows that the latter is equal to s, as is shown by taking moments of the forces shown, say about O. By reference to Fig. 8,* it is seen that s, this component of r, is given by Equation 17, and is thus identical in value with v of the formula. The unit shear, s, on a vertical plane is likewise equal to the unit shear on a horizontal plane at O, from the well-known theorem. In the demonstration, OO' was assumed to be parallel to the upper surface, which is not strictly true; in addition, V should have the meaning given later, where shear is thoroughly discussed. It can be easily shown, by a slight modification of usual methods, that the unit shear component, s (as just defined), on a plane parallel to OO' (assumed to be parallel to the surface) and at a vertical distance, y, above it, is given by the formula,

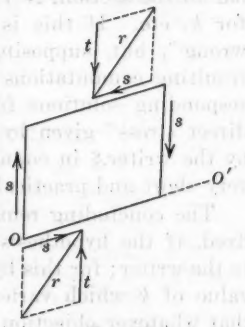

FIG. 27.

$$s = \frac{V}{b\,(k\,d_1)^2 \,.\, D\,G}\,[(k\,d_1)^2 - y_1{}^2]^2.$$

Thus, this component, s, varies according to the parabolic law, and is zero when $y_1 = k\,d_1$ or at the surface, as we know to be true from the elementary considerations given in connection with Fig. 1.†

This component, s, is thus exactly the shear on a horizontal or vertical plane at O, and "above the neutral surface, it decreases, by the usual parabolic law, to zero at N." It is not the shear on a horizontal or vertical plane between O and N. That will be given later.

It will be observed, at the point, O, that if r, Fig. 27, is resolved into components perpendicular and parallel to OO', the latter—the full shear on OO'—may be greater than s. The vertical component was

* *Loc. cit.*, p. 758.
† *Loc. cit.*, p. 746.

Mr. Cain. overlooked in the writer's first analysis. The decomposition indicates that the true shear on OO' may be greater than s.

Mr. Janni again introduces Fig. 10,* in which it seems that he intends to leave the part of the beam to the left of $I\ N$ unchanged, as MN shifts to the different positions shown by the dotted lines. As β approaches 90°, the value of f_c, given by him, becomes smaller and smaller for a constant M, and at $\beta = 90°$, $f_c = 0$, which is characterized as absurd. Messrs Slocum and Hancock† give a complete solution of this case. These authors conclude that "if a piece is bent exactly at right angles on itself, the slightest bending strain must produce incipient rupture"; also, that "the neutral axis passes through the vertex of the angle" (N in Fig. 10). This seems to dispose of Mr. Janni's "absurd conclusion."

As to the solution proposed by Mr. Janni in connection with Fig. 19, no objection can be urged, provided the inclination of the reinforcing bar to the section, $N\ I$, is properly included in deducing the formulas for k, etc. If this is not done, then the solution would be "flatly wrong"; but, supposing a correct solution to have been made, the resulting computations would be long and wearisome, as note the corresponding solutions for a prismatic beam, subjected to "flexure and direct stress" given by Turneaure and Maurer.‡ The solution given by the writer,§ in connection with the diagram for finding k and j, is very short and practical.

The concluding remarks of Mr. Janni about the neutral axis being fixed, if the hypothesis of parallel stresses is adopted, is inexplicable to the writer; for this hypothesis leads by rigorous mechanical laws to a value of k which varies with β, β_1, etc. It may be added, likewise, that whatever objection applies to the determination of k, with regard to the state of stress shown in Fig. 22 (2), would seem to apply equally to the state of stress shown in Fig. 22 (1), when β was not zero. The exact determination of O is doubtless a very complicated problem, which awaits a solution. The writer's only aim was to get an approximate result leading to safe formulas.

The writer has studied more carefully the subject of shear, since the publication of his paper, and is glad of this opportunity to state his agreement, in part, with Mr. Nishkian in his contention that the vertical component of the stress in the inclined rod should be deducted from the vertical load to get the shear in the concrete of the heel, Fig. 12.|| Thus, to take a more general case, consider the toe, Fig. 28, subjected to the vertical load, Q. The stress, C, inclined at the angle, β,

* *Loc. cit.*, p. 766.

† "Text-book on the Strength of Materials," Art. 127.

‡ "Principles of Reinforced Concrete," Arts. 80–85.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 756.

|| *Loc. cit.*, p. 768.

the tension in the rod at I inclined at the angle, β_0, to the horizontal, and the shear, V, in the section, $N\ I$, to be carried by the concrete, in addition to the vertical component of C already included. V is thus the shear in the section still to be accounted for after the vertical components of C and T have both been considered. For equilibrium, we must have, Mr. Cain.

$$V = Q - C \text{ sin. } \beta - T \text{ sin. } \beta_0.$$

This portion, $L\ M\ N\ I$, of the beam acts on the part of the beam to the right of the section, $N\ I$, with forces exactly equal and opposed to C, T, and V.

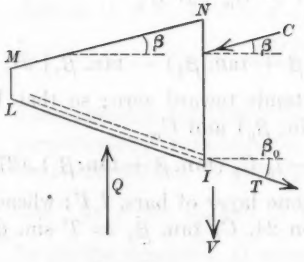

FIG. 28.

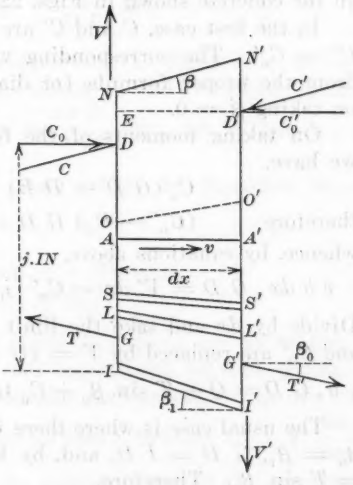

FIG. 29.

When the steel is horizontal, $\beta_0 = 0$ and $V = Q - C$ sin. β. This refers to the case shown in Fig. 22 (2). For the case shown in Fig. 22 (1), C is horizontal and $V = Q$. When T, Fig. 28, acts upward, the last term in the first formula changes sign. In the first investigation of shear, C' was assumed to act in the same line as C, which is only approximately true. To make an exact analysis, consider Fig. 29, representing a portion of the beam between the two vertical sections, $I\ N$ and $I'\ N'$, dx apart and subjected to the forces, V, V', C, and C' (the horizontal components of which are C_0, C_0'), T, the resultant of the tensions in the bars, $I\ I'$, $L\ L'$, $S\ S'$, acting at G and inclined at the angle, β_0, to the horizontal and the corresponding resultant, T', at G', supposed to act in the line of T, but opposite in direction. The constant width of the cross-sections will be called b, and the unit shear

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on a horizontal plane, $A\ A'$, just below the neutral surface, v. We have,

$$I'\ D' - I\ D = j\ (I'\ N' - I\ N) = j\ dx\ (\tan.\ \beta + \tan.\ \beta_1)$$
$$D\ E = I\ E - I\ D = I'\ D' - dx \tan.\ \beta_1 - I\ D = j\ dx\ (\tan.\ \beta + \tan.\ \beta_1) - dx \tan.\ \beta_1.$$

Now, whatever the inclinations assumed of C and C', we have

$$C_0 = T \cos.\ \beta_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$
$$C_0' - C_0 = v\ b\ dx \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

the latter equation resulting from placing the sum of the horizontal components of the forces acting on $A\ A'\ N'\ N$ equal to zero. Two cases will be considered, corresponding to the dispositions of stress in the concrete shown in Figs. 22 (1) and (2).

In the first case, C and C' are taken as horizontal; whence $C = C_0$, $C' = C_0'$. The corresponding value of j (call it j_0) can be derived from the proper formula (or diagram, for one layer of rods at $I\ I'$) on taking $\beta = 0$.

On taking moments of the forces acting on $I\ I'\ N'\ N$ about G, we have,

$$C_0'(G\ D + D\ E) - C_0\ .\ G\ D = V'\ dx \quad \ldots\ldots\ldots\ldots (26)$$

therefore $$(C_0' - C_0)\ G\ D = V'\ dx - C_0'\ .\ D\ E;$$

whence, by equations above,

$$v\ b\ dx\ .\ G\ D = V'\ dx - C_0'\ [j_0\ (\tan.\ \beta + \tan.\ \beta_1) - \tan.\ \beta_1]\ dx.$$

Divide by dx and take the limit as dx tends toward zero; so that V' and C_0' are replaced by $V = (Q - T \sin.\ \beta_0)$ and C_0.

$$v\ b\ .\ G\ D = Q - T \sin.\ \beta_0 + C_0 \tan.\ \beta_1 - j_0\ C_0\ (\tan.\ \beta + \tan.\ \beta_1)\ . \quad (27)$$

The usual case is where there is only one layer of bars, $I\ I'$; whence $\beta_0 = \beta_1$, $G\ D = I\ D$, and, by Equation 24, $C_0 \tan.\ \beta_1 = T \sin.\ \beta_1 = T \sin.\ \beta_0$. Therefore,

$$v\ b\ .\ G\ D = Q - j_0\ (C_0 \tan.\ \beta + T \sin.\ \beta_1) \quad \ldots\ldots\ldots (28)$$

where $G\ D = j_0\ .\ I\ N$.

In the second case, C and C' will be assumed to act parallel to $N\ N'$. Therefore, $C_0 = C \cos.\ \beta$, $C_0' = C' \cos.\ \beta$, and j is to be found as usual, for the given value of β. The right member of Equation 26 is now to be increased by $C' \sin.\ \beta\ .\ dx$; whence, proceeding as before, and substituting, $V = Q - T \sin.\ \beta_0 - C \sin.\ \beta$, as just explained, we have,

$$v\ b\ .\ G\ D = Q - T \sin.\ \beta_0 + C_0 \tan.\ \beta_1 - j\ C_0\ (\tan.\ \beta + \beta_1)\ . \quad (29)$$

where $C_0 = C \cos.\ \beta$. For one layer of bars $I\ I'$, $\beta_0 = \beta_1$, $G\ D = I\ D$, $C_0 \tan.\ \beta_1 = T \sin.\ \beta_0$. Therefore

$$v\ b\ .\ G\ D = Q - j\ (C \sin.\ \beta + T \sin.\ \beta_1) \quad \ldots\ldots\ldots (30)$$

where $G\ D = j\ .\ I\ N$.

In Equations 27 to 30, on dividing by $b \,.\, G\, D$, we find the shear, v. Note in the right member of Equation 30, that in place of subtracting the sum of the vertical components of C and T from Q, that only j times this amount is subtracted.* Mr. Cain.

The bond stress for the general case of many layers of bars, Fig. 29, is easily found. Using the previous notation,† and since the shear on the horizontal plane, $A\ A'$, equals the sum of the horizontal components of the bond stresses in the bars, Fig. 27,

$$b\, v\, dx = u_1\, I\, I' \cos.\, \beta_1\, \Sigma o_1 + u_2\, LL' \cos.\, \beta_2\, \Sigma o_2 + \ldots$$
$$= dx\,(u_1\, \Sigma o_1 + u_2\, \Sigma o_2 + \ldots)$$

$$\text{Therefore, } b\, v = u_1 \left[\Sigma o_1 + \frac{O\ L}{O\ I}\, \Sigma o_2 + \frac{O\ S}{O\ I}\, \Sigma o_3 + \ldots \right] \ldots\ldots\ldots (31)$$

For any one of the cases corresponding to Equations 27 to 30, substitute the value of $b\ v$ corresponding, in the foregoing equation, whence u_1, and, subsequently, $u_2 = u_1 \dfrac{O\ L}{O\ I}$, etc., can be found. For only one layer of bars, $u_1 = \dfrac{b\ v}{\Sigma o_1}$, and using, say, Equation 30, also placing $D\ G = D\ I = j \,.\, I\ N$,

$$u_1 = \frac{Q - j\,(C \sin.\, \beta + T \sin.\, \beta_1)}{j \,.\, I\, N\, \Sigma o_1} \ldots\ldots\ldots\ldots (32)$$

The formulas for shear and bond stress just given are to replace the corresponding formulas for shear and bond stress hitherto given. In the applications, it may prove desirable to express C_0 in the "first case" by $T \cos.\, \beta_0$, and in the "second case" to place $C = T \dfrac{\cos.\, \beta_0}{\cos.\, \beta}$. In all the formulas, Q represents the algebraic sum of the reaction (if any) and the loads left of the section $I\ N$. Also, if T' is directed to the right and upward, sin. β_0 is changed to $(-$ sin. $\beta_0)$. Similarly, change β_1 to $(-\ \beta_1)$ if $I\ I'$ lies above the horizontal through I, with a consequent change of sign of sin. β_1 and tan. β_1.

It will be observed that, in consequence of the term subtracted from Q being often large, the values of v and u are small as compared

* In Mörsch's "Concrete-Steel Construction" (Goodrich's translation), p. 190, similar results, for some of the simplest cases, are found by a very neat method involving the calculus. This method can easily be extended to other cases, but it is perhaps not so obvious as the above. The moment, M, of the external forces about either G or D is constant; hence, for one layer of rods, $I\ I'$, with G at I, $M = C_0 \,.\, j_0\, I\, N$ in Equation 28 $= C$ cos. $\beta \,.\, j\, I\, N$ in Equation 30; therefore $j_0\ C_0$ tan. β in Equation 28 $= j\ C$ sin. β in Equation 30. Similarly, if T_0 is put for T in Equation 28 to distinguish it from the T in Equation 30, $T_0\, j_0\, I\, N$ in Equation 28 $= T j\ I\, N$ in Equation 30; therefore $T_0\, j_0$ sin. β_1 in Equation 28 $= T\, j$ sin. β_1 in Equation 30. Whence the right members of Equations 28 and 30 are equal in value; so that the values of v derived from Equations 28 and 30 are in the ratio of the $G\ D = j \,.\, I\, N$ of Equation 30 to the $G\ D = j_0 \,.\, I\, N$ of Equation 28, or of j to j_0. The values j and j_0 differ by only a small percentage.

† *Loc. cit.*, p. 760.

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with results obtained for prismatic beams where $\beta = 0$, $\beta_1 = 0$. Hence it will rarely be necessary to use the formulas above in their entirety, since a rough estimation of the subtractive term will generally suffice to indicate safe limits.

It has been remarked, in connection with Fig. 27, that the unit shear at O, on a plane inclined to the horizontal, may be greater than the shear on the horizontal plane. The difference will usually be small. The unit shear, q, on a horizontal plane at the distance, y_1, above O in the case of the toe, Fig. 22 (1), for the state of stress shown in that figure, can be found by a method similar to that used for prismatic beams, only the rise of N' above N and of O' above O, Fig. 29, must be considered in the limits of the integrations, etc. The results are to be written only to first order infinitesimals, but even then, the work is long, so that only the final result will be given. As before, b = constant breadth and d = depth, $N\ I$, Fig. 22 (1), M = moment of external forces about I, and k_0, j_0, refer to the state of stress shown in the figure. The value of q is given by the following equation,

$$q\ (k_0^2\ j_0\ b\ d^3) =$$

$$k_0^2\ d\ (Q\ d - M \tan.\ \beta) + [2\ (1 - k_0)\ M \tan.\ \beta]\ y_1$$

$$+ \left[\frac{3\ M \tan.\ \beta}{d} - Q\right] y_1^2 \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

The unit shear at O is found by putting $y_1 = 0$ and substituting $C_0\ j_0\ d$ for M,

$$q\ (\text{at } O) = \frac{Q - j_0\ C_0 \tan.\ \beta}{b\ .\ j_0\ d},$$

as found in another manner above. At N, the shear is found by putting $y_1 = k_0\ d$. Therefore,

$$q\ (\text{at } N) = \frac{2\ M \tan.\ \beta}{k_0\ j_0\ b\ d^2} = f_0 \tan.\ \beta,$$

as can be proved independently. Here, f_0 indicates the direct bending stress at N, Fig. 22 (1).

Another check is afforded by taking $\beta = 0$, whence,

$$q = \frac{Q\ (k_0^2\ d^2 - y_1^2)}{k_0^2\ j_0\ b\ d^3}$$

which is very easily proved independently.

Reasons have been given for the computed value of f_0 being in excess, the state of horizontal stress on the vertical plane, $I\ N$, being nearer that shown in Fig. 25 than the one given in Fig. 22 (1); hence q at $N < f_0 \tan.\ \beta$, and, similarly, the variation in q is not exactly that given by the equation, as y_1 increases from zero to $O\ N$. How-

ever, accepting it as showing roughly the variation in q, it is seen that Equation 33 can be written in the form, Mr. Cain.

$$q = l + m\,y_1 + n\,y_1^2,$$

where q and y_1 are the only variables. When $n = 0$, this is the equation of a straight line. When n is not zero, it may be positive or negative, and the equation is that of a parabola, the axis of which is horizontal, and the vertex is at the point, the abscissa of which (parallel to axis of q) is $\left(l - \frac{m^2}{4\,n}\right)$ and the ordinate (parallel to axis of y_1) is $\left(-\frac{m}{n}\right)$.

Figs. 30 and 31 show the loci for $n > 0$, $n < 0$, respectively, for assumed values, the value of q at $y_1 = O\,N$, being f_0 tan. β in both cases. In Fig. 30, q increases with y_1 from O to N; in Fig. 31, q decreases with y_1 above the vertex. Of course, only the portions of the curves from $y_1 = 0$ to $y_1 = O\,N$ pertain to the problem. If the load, Q, acts a units to the left of $I\,N$, Fig. 22 (1), we have, $M = Q\,a$, hence,

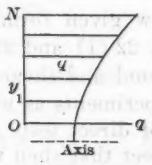

FIG. 30.

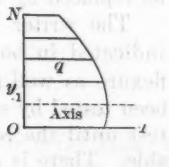

FIG. 31.

$$l = \frac{Q\,(d - a \text{ tan. } \beta)}{j_0\,b\,d^2}$$

is always positive, since $d > a$ tan. β. In fact, when $y_1 = 0$, $q = l$, or the shear at the origin acts to the right, so that the locus, whether a parabola or a straight line, cuts the axis of q to the right of O. It is evident from the general equation above that m is always positive. Also, since

$$n = \frac{Q}{k_0^2\,j_0\,b\,d^3}\left(\frac{3\,a \text{ tan. } \beta}{d} - 1\right).$$

it is evident that,

$$n > 0, = 0, < 0,$$

according as,

$$a \text{ tan. } \beta > \frac{d}{3}, = \frac{d}{3}, < \frac{d}{3},$$

corresponding, respectively, to a long toe, a medium toe, or a very short toe. When n is positive, the abscissa of the vertex may be positive, as shown in Fig. 30, or it may be negative. When n approaches zero, the vertex recedes indefinitely to the left and downward, and the locus approaches the straight line, $q = l + m\,y_1$, where m and l are always positive.

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Similarly, when n is negative, Fig. 31, although the vertex of the parabola is always in the first quadrant, yet as n tends toward zero, it recedes indefinitely from the origin, and the shear increases in going from O to N, and the locus approaches again the straight line, $q = l + m\, y_1$.

The foregoing general formula only gives the unit shear, q, on a horizontal (or vertical) plane at the distance, y_1, above O. At this point, in addition to these shears, there is a horizontal pressure, f, on a vertical plane and a vertical pressure on a horizontal plane. The latter is unknown, except at the surface, $M\ N$, where it is $f_0 \tan.^2 \beta$, as can easily be proved by aid of an infinitesimal wedge like $A\ M\ B$, Fig. 23.

If the state of stress indicated by Fig. 22 (2) is assumed, the same general formula, as given above, is derived, except that k_0, j_0, are to be replaced by k, j.

The writer has now given formulas for the two states of stress indicated in both Fig. 22 (1) and 22 (2), referring to stresses due to flexure as well as to bond and shear stresses, and certain results have been tested by such experiments as were available. The case must now rest until the results of direct tests on wedge-shaped beams are available. There is a prospect that such tests will be made at several institutions within the next year.

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PAPERS AND DISCUSSIONS

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GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

By Harold A. Rands, Assoc. M. Am. Soc. C. E.†

Mr. Rands.

Harold A. Rands,‡ Assoc. M. Am. Soc. C. E. (by letter).—This paper has brought forth expressions and observations from members of the Profession who have had experience in grouting, and, although it was hoped that others would be heard from, it is believed that, in a general way, the discussions cover the knowledge concerning this subject at the present time. In addition to the structures cited in the paper and the discussion, in the construction of which grouting has been resorted to, mention may be made of the Hale's Bar Dam, in Tennessee, the Houlter Dam, in Montana, the Kananaskis Dam, in Alberta, and the Municipal Dam of the City of Seattle.

Possibly what might be the most interesting and instructive case of all is that of a high earth dam, the fill for which was commenced without due regard for obtaining a tight cut-off. After the dam was completed, and the pond filled, the leakage assumed serious proportions, and the back portion of the dam itself began to slough away. Holes were put down at various points through the dam and into the underlying material, and grouting was resorted to. It is reported that a total of 30 000 bbl. of cement, in the form of grout, was poured or pumped into this dam, though, unfortunately for the Profession, the "soft pedal" was put on the engineers in charge of the work, and the facts were never published.

This dam is mentioned here as showing the importance of careful attention to the blind cut-off of an earth dam. In contrast with the lack of consideration in this case, attention is called to the Lahontan

* Continued from August, 1914, *Proceedings*.
† Author's closure.
‡ Oregon City, Ore.

Mr. Rands.

Dam, in Nevada, which is also of earth, and beneath which the engineers of the Reclamation Service specified a concrete cut-off, 30 ft. deep, with a grouted cut-off going down an additional 30 ft., or 60 ft. in all.

Considering the contributions from the engineers connected with the Estacada Dam, we have first the discussion by Mr. Rippey. In the earlier paragraphs of his discussion, Mr. Rippey expresses fear lest the writer's views be given too much weight, and sums up the reasons for his somewhat extended discussion, in the following:

"In order, therefore, that the results obtained at Estacada, and the conclusions drawn therefrom by Mr. Rands, may be properly appraised by others interested in the treatment of deficient foundations, it is proper to outline, as briefly as possible, sufficient of the history of that project to indicate the unfavorable character of the governing conditions under which the work was executed."

Mr. Rippey then proceeds to attribute the comparatively poor showing made by the grouting at Estacada to "lack of co-operation" on the part of the local organization of the company owning the dam.

The writer went to considerable pains to avoid mentioning the "Chapters of Grief" which unfortunately marked the construction of the Estacada Dam, believing that the Profession at large is only concerned in such affairs when they affect results. In this particular case, it was his belief that the great amount of detailed information, together with the tables and curves submitted, constituted fairly acceptable evidence as to the care, and the regard to detail, with which the grouting was carried out.

However, since the publication of Mr. Rippey's part in the discussion, the author has received a letter from one of the foremost hydraulic engineers in the United States, wherein is expressed the opinion that grouting, to be successful, must be done with close attention and care in every detail. As this sentiment, in part, may be the result of the reading of Mr. Rippey's discussion, and possibly is shared by others, it may be proper to cover in a paragraph the history of the Estacada Dam.

During the preliminary investigation and the first three months of the construction period, the work was completely under the direction of the local organization, with Messrs. Sellers and Rippey acting as Consulting Engineers; during this period no grouting was done. During the next five months—or what may be called the middle portion of the construction period—the work was under the direction of the Sellers and Rippey organization, with Mr. William H. Cushman in charge as Resident Engineer; it was during this time that the grouting was commenced. During the remainder of the construction, the work was under the direction of the local organization, with Mr. Frank R. Fisher in charge as Resident Engineer, and with Messrs. Sellers and Rippey continuing as Consulting Engineers. The construction force, it should

be said, was under the direction of the contractors, the Puget Sound Bridge and Dredging Company, except as to the grouting, which was directed at all times and as to all details by the Resident Engineer. Mr. Rands.

The change in the middle of the work, which came as a result of conflict in organizations in a job literally loaded down with organizations, and not from anything which in any way reflects on either the integrity or engineering ability of any one, in place of affecting the grouting injuriously, really amounted to giving it a second trial, with every added care and refinement of detail which it was possible to devise. In fact, so poor was the showing being made by the grouting, both at Estacada and at the site of the proposed 150-ft. dam, at the time of the second change of management, that it is probable it would have been discontinued altogether, but for this change. Both Mr. Cushman and Mr. Fisher, it may be added, have reputations for thoroughness and attention to detail.

As to the writer's personal opinions and theories, though they might be subject to liberal discount, were they based wholly on the somewhat negative results at Estacada, they may be entitled to some additional consideration by reason of their having been tried out on another grouting job which proved more successful.

Soon after the completion of the Estacada Dam, the writer was engaged by E. G. Hopson, M. Am. Soc. C. E., Supervising Engineer of the Reclamation Service, to go to Nevada, and take charge, under D. W. Cole and F. H. Tillinghast, Members, Am. Soc. C. E., Project Engineer and Resident Engineer, respectively, of the grouting of the Lahontan Dam. This work has been described fully by Mr. Cole in articles appearing in the engineering papers, and it is interesting to contrast the results obtained in the poorer or left-channel section at Estacada with those in the poorer or second section at Lahontan.

At Estacada, the final tests showed an average leakage, at full depth, of 36 gal. per min., or 75% of the leakage of the primary holes at the same point; at Lahontan, it was only 6 gal. per min., or 9% of the leakage of the primary holes. At Estacada, a very small head of water against the coffer-dam produced, even in the final holes, a very heavy back-flow; at Lahontan, all back-flow ceased before the final holes were put down. At Estacada, communication between the seventeen final proving holes was so free that a pressure slightly exceeding the head to be produced by the dam applied to one of them caused water to flow from all the others; at Lahontan, the grouting of all the holes as soon as drilled prevented the observance of anything directly comparable to this, yet, at times, a second hole would be down to nearly full depth only 12 ft. from a hole being tested, and the fact that no water was forced from the former hole shows that there was no communication, as at Estacada.

Mr. Rands.

At Lahontan, it may be stated, there was evidence of a change in the structure of the underlying material, in that in the earlier holes put down there was much trouble from caving, and binding of the drills in the holes, in some cases causing the loss of an entire shift, and even of the bits, in getting the rods up and the caved material re-drilled; in the drilling of the final holes, no such trouble was experienced, the whole number being put down without a drill sticking or a single cave-in.

As the writer takes little blame to himself for the relatively poor showing at Estacada, so does he take little credit to himself for the relatively good showing made at Lahontan. Though experience taught him much during the grouting of the 34 000 ft. of drilling at Estacada, there was not enough difference between the methods used on that work and at Lahontan to account for the difference in results. The real reason, in the writer's opinion, is one of geology, and therein, and not in the lack of sympathetic co-operation, lies the vindication of Mr. Rippey.

Referring again to the paper by Mr. Schreiber, read before the Western Cement Users Association, and wherein is set forth what Mr. Rippey hoped for in recommending grouting for the Estacada Dam (page 10*), it will be noted that the recommendation concludes with the following:

"It [the sufficiency of the grouting] must be susceptible of demonstration by actual test before the superstructures should be started."

At Estacada it was tried with indifferent success, due to the geological formation; at Lahontan it was tried with positive success, again due to the geological formation.

It was with no spirit of irony that the writer, in the paper, stated that the credit for evolving the grouting programme for the Estacada Dam belonged to Mr. Rippey, and it is with no spirit of irony that it is repeated. He not only proposed the programme for this work, but what is more, he gave the Profession the only safe rule to be followed in cases where grouting is to be resorted to in lieu of the usual concrete cut-off.

The proposal of Mr. Hulse to resort to slow sedimentation in lieu of the more rapid pressure grouting is a real addition to the subject, though, in the writer's opinion, this should not be resorted to until the latter had failed to produce results. The writer still believes that to continue almost indefinitely the pouring of thin cement grout into a hole is, in a great measure, to waste the cement, for, to quote Mr. White:

"It may run a long way from the part to be grouted, and on excavating this part, very little of it will be found. No one knows where it went—all that is known is that one has paid for it."

* *Proceedings*, Am. Soc. C. E., for January, 1914.

Mr. Hulse does not believe that it is safe to blast a trench in the material on which it is proposed to erect a great dam, and expresses himself as follows: Mr. Randa.

"If it [the trenching] is done by blasting, * * * there is great danger of doing almost irreparable damage to the surrounding material, and this is exactly what happened in the left channel at Estacada, where the breccia adjoining the heel-trench was shaken and cracked in all directions by the blasting."

The writer believes that this statement is far too strong, and may leave the impression that the Estacada Dam, because of the blasting for the cut-off, is standing on unsafe and insecure foundations.

It is admitted that the blasting for the cut-off trench was, in the beginning, ignorantly done; and, to support this statement, it is only necessary to say that twenty-one vertical surface holes were loaded and fired at one time. The result of this was to loosen up a large quantity of material which had to be replaced with concrete. By referring to Fig. 4, the flaring cross-section of the trench in the left channel shows the result of this method of excavation. Mr. Cushman, who arrived at the dam site before any concrete was placed, gave this matter the closest attention, had all the loosened material removed, and the bottom of the trench hand-picked to a depth of from 1 to 2 ft. below the effect of any shots. That the blasting had nothing to do with the free leakage of the holes in the left channel is readily seen by referring to the tabulation on page 23,* or to the curves on Fig. 10, which indicate that the leakage, even of the primary holes, was insignificant until a depth of 30 ft. below the cut-off had been reached.

With a competent powder man in charge as foreman, there would seem to be little danger of doing irreparable damage to the surrounding rock, and against the more than one able engineer, whom Mr. Hulse states that he has heard argue against the practice, the following editorial comment† regarding the grouting at Lahontan, is quoted:

"On this dam the estimate of the saving made was only $3 000, compared with the cost of trenching and filling with concrete to the same depth. Many engineers would prefer the trenching method where the difference is so small."

In justice to the engineers of the Reclamation Service, it should be stated that it was the saving of time rather than money, which led to the adoption of the grouting programme, which, as the cut-off was already 30 ft. deep, really amounted to making assurance doubly sure, or super-insurance against leakage.

As to the general proposition of trusting to the grout to form a

* *Loc. cit.*

† *Engineering News.*

Mr. Rands.

cut-off, even to the closing of surface seams, in lieu of the usual concrete-filled trench, there is certainly room for difference of opinion. A case in point is that of the dam at Austin, Pa.

The failure of this dam, in the start at least, seems to have been due to the slipping or sliding of the immediate stratum, on which the dam stood, over the stratum next below, the pressure of the water in the pond having changed the clay in the seam between the strata into an excellent lubricant. A concrete cut-off would undoubtedly have prevented the catastrophe, though it is hard to see how any amount of grouting could have altered the unfortunate result.

Mr. Fisher rather objects to the writer's expression to the effect that "most engineers will conclude that over a portion of the cut-off no grouting was needed, and that over other parts where it was needed, it did little good." This, of course, is only a matter of opinion, though it may be noted that Mr. Cushman, who is equally conversant with the Estacada conditions, considers the attempt there to have been, "to a large extent, futile."

Mr. Fisher also states that the engineers should be given credit for what was accomplished. The writer had no desire to take credit from any one, but he did feel that the Profession was entitled to the full facts concerning the Estacada grouting. Whether the grouting was carried to an unwarranted length and expense is again merely a matter of opinion; whether the method is good or bad, as applied to another structure, may be vital.

To Mr. Cushman belongs the credit for suggesting the possible shattering of the material between the drill holes in order that it might receive thick grout and form an effective cut-off, though this was not tried at Estacada, nor, so far as known, on any other work; to Mr. Cushman also is due the credit for suggesting the capitalizing of the loss through seepage and balancing it against the cost of grouting. Probably, also, more than he is aware, the writer is indebted to Mr. Cushman for ideas throughout the paper.

Mr. Coburn states it as his belief that the natural sedimentation, possibly accelerated by a deposit of suitable material above the dam, would have effectually sealed the foundation material without resorting to the grouting treatment. In reference to this, it may be of interest to consider the Crowley Creek Dam, a very thin concrete arch structure wedged between the rock walls of a box canyon in Malheur County, Oregon. This dam was designed by A. J. Wiley, M. Am. Soc. C. E., and concerning it, the following is quoted from the Third Biennial Report of the Oregon State Engineer:

"During the first season's use there was a very perceptible loss of water which passed through seams in the lava and re-appeared below the dam, but this leakage has gradually lessened and is no longer of any moment."

Mr. Rands.

Considering those portions of the discussion contributed by members not connected with the Estacada Dam, there is much of value in those of Mr. Abbot and Mr. Souder, the former covering the grouting of Lock and Dam No. 2 in the Mississippi between St. Paul and Minneapolis, and the latter that of the Hinckston Run Dam, in Pennsylvania. These seem to be the pioneer examples of grouting in America, and it is gratifying that they are now so fully made a matter of record.

In his discussion, and in his earlier excellent paper on the Fremantle Graving Dock,* Mr. Ramsbotham has given a view of grouting from another angle, and he, too, because of designing his own grouting machine, deserves to be classed among the pioneers.

Mr. White has summed up the subject so fully in accord with the writer's idea, and in so much better and so much more concise language, that to comment would be to detract.

Mr. Paul's description of the grouting of the great Arrowrock Dam, and the conclusions drawn therefrom, are valuable evidence. It is interesting to note how nearly parallel Mr. Rippey's original recommendation as to the necessity of "actual demonstration in advance of construction" is to Mr. Paul's final injunction to the effect that "where grouting is actually needed, it is unsafe to depend on it, except after convincing evidence, at the site, that it will give the results desired."

It is not proper to close this discussion without some additional statement as to the safety of the Estacada Dam. As has been said, no springs have appeared below the dam as a result of its construction, and the fact that the pressure of the water in the hole put down inside the dam was sufficient to cause the water to rise within 13 ft. of the level of the water in the pond, is good evidence that no seepage or leakage has occurred. It is true that there is the chance that the water under so great a pressure may find an outlet, but, as time goes on, and the natural silting processes take place, the danger of this becomes less likely.

The type of dam renders negligible any danger from uplift, though it may be pointed out that no seepage was encountered from the holes put down inside the dam until more than 20 ft. of rock had been drilled through, and it was not until a depth of 60 ft. of rock had been reached that the pressure became sufficient to raise the water to the height noted. The weight of this rock, plus the weight of the dam resting on the buttresses with 5-ft. footings, spaced 18 ft. from center to center, plus the resultant of the water pressure acting on the 45° deck, effectually negatived any upward pressure from the water under the dam. It is perhaps unfortunate that the term "leakage" has been applied to the water which it has been possible to "lose" through the holes in testing. For, as Mr. Fisher has pointed out, the conditions are much more favorable for leakage or loss of water, through

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1942.

Mr. Rands.

the rock with a 50-ft. hole penetrating its interior, than when later the pond is filled and no such penetration exists. Also, as Mr. Fisher has pointed out, the leakage appears to be small indeed when expressed in second-feet, or rather as a fraction of a second-foot, instead of gallons per minute.

The writer feels that this explanation is due in order to counteract any inference that might be gained to the effect that, because of the failure of the grouting to do all that was hoped for it, the dam is unsafe.

In conversation, Mr. Coburn, shortly after the appearance of the paper and the first of the discussions, said that, in his opinion, the question of grouting appeared to be just where it was before, that is, it is still incumbent on those advocating grouting to do all that they claim for it to prove their point; and this seems to be the case.

Consider the dams that have either partly or entirely failed during the last few years in America. We have the Austin, Pa.; the Austin, Tex.; the Pittsfield, Mass.; the Port Angeles, Wash.; and the Stoney River, West Virginia. Of these, that at Austin, Pa., has already been considered. The dam at Austin, Tex., though, according to the late Joseph P. Frizell,* M. Am. Soc. C. E., stood on material in which "lay strata hard and soft", and though there was a single leak under the dam from the time of its completion to the day of its failure, through which flowed 6 000 000 gal. of water every 24 hours, the real cause of the failure seems to have been due to the undermining effect of the water at the down-stream toe, which, months before the dam failed, had progressed so far that bottom could not be touched with a 10-ft. pole. Had the apron been added, as recommended by the engineers, the dam in all probability would not have failed. Some may say that the lack of the apron was only one of the causes which produced the failure, and that, had some means been used to seal the material effectually along the up-stream heel, the uplift pressure would have been much reduced and the dam would not have failed at the time it did. In reply, it is only necessary to point out that grouting has not yet shown its adaptability for correcting "soft strata".

The other three dams mentioned, namely, the Pittsfield, the Port Angeles, and the Stoney River, all failed from one and the same cause—failure to carry the excavation to bed-rock. Grouting would probably not have put off their failure by so much as one day, though all might have been saved by a proper back-fill, as was suggested by Mr. Coburn for Estacada, which treatment has been most thoroughly covered in several articles by Mr. G. W. Bligh, appearing at various times in the engineering papers.†

* "Water Power," by Joseph P. Frizell, 3d ed., p. 245.

† *Engineering News*, December 29th, 1910, p. 708; February 6th, 1913, p. 226.

Mr. Rands.

The real trouble with grouting is that it is too easy to think about doing; too easy, when a stratum of poor material is unexpectedly encountered in making the excavation, to set in a few pipes and go ahead with the construction, trusting to the grouting to make it right—a practice on a par with the oft-repeated expedient of putting an additional nail or two in the end of a board because of a knot or a rotten spot in the middle.

Even admitting that, because of lack of sympathetic co-operation or from any other cause, the grouting at Estacada was not done as well as it should have been, the fact that a colossal attempt, costing nearly $70 000, resulted in indifferent success, goes a long way toward proving that grouting is far from being yet a very strong competitor of the conventional cut-off. Or, considered at its best, it seems a far call from any results that it has yet achieved to the day when any material requiring treatment to carry the load of a great dam, and make it safe against the tireless, insistent, searching pressure of the water, can be rendered secure by grouting.

It is true that in the discussion mention is made of a certain foreign dam, the name and location of which the writer was not at liberty to give, wherein the grouting had been a "complete success". It is also true that those who advocate this system of foundation treatment qualify their recommendations with the provision that the sufficiency of the grouting to do the work must be demonstrated in advance of actual construction. It must be remembered, however, that only sometimes are recommendations acted on, and that even the Estacada Dam, for the benefit of which this rule was first proposed, because of the insistent demands of the owners for early power, was made an exception thereto.

The writer still believes that grouting has a legitimate field in preventing possible seepage and in solidifying the material under a solid dam in order that the uplift due to water passing under the concrete cut-off may be reduced. In short, as an agent to "make assurance doubly sure".

He regards as premature and unwarranted, by anything it has yet achieved, to claim for the grouting process the formation, in any but the most favorable material, of an effective cut-off. The real danger lies in dams of 40 to 75 ft. in height, the construction of which may be undertaken by companies of limited means, and, in the design and supervision of which, engineers of limited experience may be engaged. It is in the construction of these dams that there is danger in expecting too much of grouting, or, as Mr. White puts it, in making "a sort of fetish" of it.

The result of this, when poor material is encountered and an insistent and impatient owner is to be satisfied, will be to set a few grout casings, but the sequel may be disaster and death.

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PAPERS AND DISCUSSIONS

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THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

Discussion.*

BY MESSRS. JAMES ALEXANDER SMITH AND F. LAVIS.†

Mr. Smith.

JAMES ALEXANDER SMITH,‡ ESQ. (by letter).—The matter of the gauge of railways is of peculiar interest in Australia. It has been claimed, as an argument in favor of the 4 ft. 8½-in. gauge, that Australia has adopted this width, but this is not so. Some misunderstanding may have been caused by official publications issued by the Commonwealth Railway offices. These publications have been the subject of a Parliamentary enquiry, and it has been shown that they are inaccurate, and that the signatures of State Engineers, who have been consulted, are appended to matters in respect to which they did not concur, and that their protests were suppressed.

The writer suggests that, in the interests of accuracy, it be recorded that Australia has not, either in respect to the several States or the Federal Government, adopted the 4 ft. 8½-in. or any other uniform gauge. The facts, at the present date, are as follows:

Early in 1914 a conference of the Premiers of the several States —which control all the Australian railways in operation—and of the Commonwealth Prime Minister, was convened and held, to deal finally with the question of a uniform gauge for Australia.

Several of the States favored the appointment of an independent commission of railway experts, representing Great Britain, Europe and America.

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* Continued from September, 1914, *Proceedings*.

† Author's closure.

‡ Melbourne, Victoria, Australia.

Mr. Smith. The Prime Minister advocated the remission of the question to a lay "Interstate Commission," recently appointed to advise regarding trade and fiscal matters affecting more than one State. This was agreed to, and the conference disbanded.

Some months later, when interrogated in the Federal House, the Prime Minister stated that he had found that, under the Constitution, the Commission had no jurisdiction unless specially authorized by a vote of the House, therefore that the action just mentioned had been *ultra vires*. He also stated that he did not propose to seek the authority of the House, which has since been dissolved.

In the interim the Commonwealth Government has decided to reticulate the whole of the great "Northern Territory" with railways of 3 ft. 6-in. gauge. That is also to be adopted for the North to South Trans-Continental. Commitments on that basis are being entered into.

Simultaneously, the Commonwealth is constructing the East to West Trans-Continental on a 4 ft. 8½-in. gauge. This road connects, to the East, with the 5 ft. 3-in. system of South Australia and Victoria; and, on the West, in default of a specific arrangement by the Commonwealth as to gauge and financial support, Western Australia is preparing to construct its great extension of the Trans-Continental to the Coast on the 3 ft. 6-in. gauge.

The position is most deplorable, and indicates that unity of gauge is anything but an accomplished fact in Australia.

Mr. Lavis. F. Lavis,* M. Am. Soc. C. E. (by letter).—Some surprise has been expressed to the writer by certain engineers, who have had much experience in railroad construction, as to the necessity for any such elaborate discussion of the gauge question at this late day, and, even supposing the question to be of interest elsewhere, why the matter should be thought of sufficient interest to warrant the publication of such an elaborate paper by this Society.

Of course, the latter criticism needs no argument for its defence, the Society is too large and important to confine its activities necessarily to matters of concern only to the United States, and the great interest which has been developed by the citizens of this country in South America since the presentation of the paper is ample justification, if any be needed, for the publication of any matter of interest in regard to the development of this vast area, in which the United States, and incidentally the engineers of this country, are bound to play an important part.

In regard to the first criticism, the writer admits that it must seem strange to American engineers, or to any others with knowledge of American railway methods, that it was considered necessary to go into so much detail in regard to many things which in the United

* New York City.

States are accepted as axiomatic. A study, however, of the voluminous discussions of the past ten years and up to the present, in regard to the gauge question in India and Australia, two countries where the matter is now of vital interest, will show that most of the fallacies in regard to gauge—if not all—which we believed long ago disproved, are still held by many engineers as well as laymen by no means obscure. The persistence of the curve fallacy, as shown by some of the discussions, in spite of the concrete examples given in the paper, is only one instance of this. Mr. Lavis.

The present status of the gauge question in Australia is shown by Mr. Smith's discussion, which apparently indicates that in spite of a former decision to do all that was possible to unify the gauges, feverish activity seems to have developed in just the opposite direction, namely, to build as many lines as possible of different gauge.

Almost coincidently with the presentation of this paper, Sir Guilford L. Molesworth, one of the most prominent engineers connected with the administration of Indian railways, read a paper* urging the importance of the immediate consideration of some measures looking to the ultimate unification of the gauges of that country, but complicated by this question of gauge.

The present temporary stagnation of railway construction in South America is bound to be followed sooner or later by an era of activity. It seems to be probable that North American engineers will have a larger voice in these matters than they have had hitherto, and this seems especially appropriate inasmuch as the problems to be overcome are largely those which they have had to solve in the United States and Canada.

The writer is quite willing to admit that the criticism advanced by Mr. Henry is to some extent well taken, and that the estimates used in the paper of the difference in cost of construction of new lines due solely to difference of gauge, are "liberal" in so far as they exaggerate, if anything, the relatively higher cost of the wider gauge; but, inasmuch as these estimates were made with a view of their probable application to actual construction in a foreign country, it was hardly thought desirable to afford opportunity for criticism on the opposite side.

The problem of deciding between the narrow-gauge and 4 ft. 8½-in., or wider, does not seem to the writer to admit of much further discussion or doubt, but the matter of deciding between the 4 ft. 8½-in. and the 5 ft. 6-in., is one of considerable difficulty.

If the Argentine alone is considered, of course, it seems desirable for many reasons to adopt the 5 ft. 6-in., not because it has any greater capacity, but because it is established, and, for new extensions, its

* "The Battle of the Gauges in India," *Journal*, East India Assoc., London, England, April, 1914.

Mr. Lavis.

additional cost over the 4 ft. 8½-in. would be comparatively little. Mr. Hill's opinion in regard to this is worth noting; though a lowered center of gravity, as has been shown by the effect on track of some of the electric locomotives first designed with very lower centers of gravity, is not an unmixed blessing.

The writer's study of the possibility of applying some scheme of unification of gauge to the whole of that portion of South America south of the Valley of the Amazon (or even including part of the southern water-shed), on account of the political subdivisions and the apparent difficulty of coming to any agreement between so many diverse interests, may be considered rather fanciful. The fact, however, that as long ago as 1890, a commission headed by as eminent an engineer and railroad man as the late Alexander J. Cassatt, and working under an agreement participated in by practically every country in the Western Hemisphere, investigated the possibilities of constructing a line, or series of lines, of railway linking up at least all the capital cities, shows that there has been for many years, in the minds of far-seeing men, a vision of a system of railroads covering the two continents, which will permit rapid and easy inter-communication between all parts. Considering, therefore, this vast area south of the Amazon and east of the Cordilleras as a single geographical unit to be covered by a network of railways, all of the same gauge and suited to modern conditions of transportation, such a system can hardly be regarded as entirely visionary.

Whether it be possible finally to reach an agreement for this purpose is difficult to determine, but the writer believes that if it may be, that, for reasons given, the 4 ft. 8½-in. gauge, as Mr. Hill states was true of the United States, is all that the "traffic of the country could bear". The idea that the constructors of the broad-gauge lines of the Argentine would not sit passively by without attempting to throw out extensions toward the north is one of course to be considered. The broad-gauge lines of the Argentine are to-day the most important group of transportation lines south of the United States, but this is only one item in a very complex problem which would have to be worked out to accomplish a desirable result, and, though not impossible of solution, it will require more than the efforts of any one man. The writer can only hope to have called attention to the problem and to some of the factors which will have to be considered in its solution, if a solution is ever desired.

The discussion by Mr. Corry is interesting in view of his long experience with the operation of the standard-gauge mountain roads of Southern Peru. Of course, it is true that railways should be built—as E. H. McHenry, M. Am. Soc. C. E., put it a long time ago—in such a way that the dollars spent for their construction will earn the most interest, and it is easily conceivable, and is admitted in the paper,

that there are many mountain districts where narrow-gauge lines to be used as feeders may be built to advantage. Mr. Lavis.

Mr. Corry shows that, on the standard-gauge lines of the Southern Peruvian Railway, curves of 328 ft. radius are successfully operated, and states that it is debatable whether it would have been advisable to have built the line with a gauge of 3 ft., using curves of 265 ft. radius. He states that it is axiomatic "that a considerable amount of traffic, even in a mountainous country, can be handled at less cost per ton per mile on a standard gauge than on either of the narrower gauges, and the standard is the more economical even if it does cost more to construct."

Yet Mr. Corry questions the advantages of the standard-gauge for mountainous regions, for the reason that sharper curves can be used and thus the graduation item can be reduced. This, in spite of the fact, as stated in the paper, that locomotives weighing 850 000 lb. are operated on the standard-gauge line of the main line of the Atchison, Topeka and Santa Fé Railroad on curves of 357 ft. (109 m.) radius, and the citation of the traffic handled on curves of 100 ft. radius on the rapid transit lines of New York City. For regions which may be served by lines of less than meter-gauge, the possibility of the use of excessively sharp curvature may perhaps be at times a controlling feature, but such lines can hardly ever be considered as main lines of transportation, and though they have their uses, and the Antofagasta is an excellent example of the value of 2 ft. 6-in. gauge, though it is now being forced to widen it, and will eventually be forced to widen again or double-track, they hardly enter into the consideration of the development of the transportation system of a continent.

Mr. Corry's illustration of the difference in cost between a line of 3-ft. gauge with 3% grades and curves of 120 m. radius, and one of 2 ft. 6-in. gauge with 5% grades and curves of 50 m. radius, can hardly be considered as having any bearing on the question of the difference in gauge, even as between 2 ft. 6-in. and 3-ft., inasmuch as the difference between the 3% and 5% grades might easily have caused all the difference in estimated cost.

It would be extremely foolish, of course, to say that standard-gauge, either 4 ft. 8½-in., or 5 ft. 6-in., should be used everywhere, and under every conceivable condition, and there is no question at all that Mr. Corry is right in pointing out that there are some places where it may be advisable to use a narrower gauge; but, as he also intimates, there is often an ultimate economy, by reason of the reduced operating expense of the wider gauge, even (and the writer believes, often more especially) on mountain lines.

Mr. Foster also seems to believe, at least to some extent, in the benefits to be derived from the possibilities of using sharper curves for narrow-gauge. This, perhaps, is due to some extent to the limitations

Mr. Lavis.

imposed on the 5 ft. 6-in. lines of the Argentine, by reason of the construction of their rolling stock, as well as by Government regulations. Perhaps, however, when opportunity permits the further consideration, by the Government and the railways of the Argentine, of the desirability and necessity of the use of automatic couplers, this will eventually require the abolition of the side-buffers and the use of a coupling which will—as it does in the United States—permit the operation of much heavier rolling stock over quite as sharp curvature as that in use on the meter-gauge lines of the Argentine. It is true that, on the meter-gauge, well-maintained curves of 3 300 ft. radius can be taken comfortably at full speed, but this is also true of the 4 ft. 8½-in. gauge, and at much higher speeds, as shown by actual practice, and the writer believes that it would be equally true of the 5 ft. 6-in. gauge, with rolling stock of the type in general use in the United States. In regard to turn-outs for the use of main-line trains, and for general use in yards occupied by passenger trains, there has been a general trend in the United States toward the use of No. 8 frogs (or 1 in 8) as a standard, in preference to those of greater angle.

Mr. Osborne evidently has not read the paper carefully if he understands, as he states, that the writer claims that the 4 ft. 8½-in. gauge in the United States and Canada shows the railway systems of those countries to be the most efficient in existence, and that this efficiency is due largely if not entirely to their gauge.

The mere fact that the writer suggests the adoption of the 5 ft. 6-in. gauge, when considering the Argentine alone, is sufficient to show that he considers efficiency with this gauge entirely possible and practical. All that is said in the paper, indeed, is that the 5 ft. 6-in., though costing more to build, offers no advantages as a transportation machine that the 4 ft. 8½-in. does not possess, nor does it permit of cheaper operation. There are doubtless many individual cases of railroads on which traffic is handled at a cost even lower than the average of the United States, but, speaking generally, it is true that, taking the country as a whole and considering the volume of traffic handled, the transportation of bulk freight is done at a lower cost here than elsewhere. If Mr. Osborne will read the paper carefully, however, he will note that it is claimed that this is due principally to the ability to handle very heavy trains, and has been brought about very largely by the increase in average train loading, in spite of great increases in cost of wages, etc. There is no claim in the paper that this economy, or perhaps more, might not have been attained on the 5 ft. 6-in. gauge, though, most assuredly, it could not have been attained on the meter-gauge. The claim is, however, that the extra cost of the 5 ft. 6-in. is not warranted, as it is not compensated for by any decrease in operating cost or any advantage over the 4 ft. 8½-in.

Mr. Lavis.

Mr. Osborne is quite mistaken in assuming that the area south of the water-shed of the Amazon is to any large degree within the tropics. Rio, which is taken as approximately the northern limit at the coast line, is just within the tropics, and though the interior part of the area under consideration runs somewhat farther north (that is, toward the equator), it is at an elevation of 2 000 ft. or more, and has a salubrious climate.

It is entirely improbable, as Mr. Osborne states, that the traffic of this vast area will ever approach the density of that of the United States, inasmuch as it is improbable that manufactured articles will be produced in it in any quantity, but nearly all the traffic is bulky and can only be hauled long distances cheaply—and it must be handled cheaply to move it—in cars of large capacity in comparatively heavy trains. The bulky agricultural traffic of India has been given by more than one prominent engineer and railway official as the reason for the adoption of the 5 ft. 6-in. gauge there, and though undoubtedly it is possible to handle heavier equipment and trains on the 5 ft. 6-in. than on the 4 ft. 8½-in., it is a fact that it is not done, so that it is considered that the 4 ft. 8½-in. is sufficient for these countries, and there is nothing to be gained by using the 5 ft. 6-in.

Time and time again the engineers and military authorities of India have advocated what Mr. Osborne does, that if it is not possible to unify the gauge, each gauge should be confined to a definite area; this has always been the cry, but it has never proved practical to do it, and the meter-gauge lines of Northern India are to-day agitating the matter of obtaining their own outlet to the coast, and Mr. Smith's testimony shows the effect in Australia. It is quite possible, of course, to admit—with Mr. Osborne—that under certain conditions all gauges are good and capable of efficient operation, but it is undoubtedly the consensus of opinion, of railway operators as well as of engineers, that the broad-gauge is a far more efficient transportation machine than the narrow-gauge, and enough more so in almost every instance to compensate for the comparatively very slight additional cost of construction, and as between the 4 ft. 8½-in. and 5 ft. 6-in. gauges, there is no particle of evidence to show that the former is not as efficient as the latter.

It is a fact that the 4 ft. 8½-in. gauge has served its purpose admirably, the effect of the introduction of a wider gauge anywhere has almost invariably been the subsequent introduction of the narrow-gauge, therefore, the writer feels that he is not only following well-established precedent in advocating the 4 ft. 8½-in. as the standard, if it be possible to unify the gauges of Southern South America, but also that this precedent is founded on wide practical experience.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

Discussion.*

By W. J. Douglas, M. Am. Soc. C. E.

Mr. Douglas.

W. J. Douglas,† M. Am. Soc. C. E. (by letter).—There have recently been completed, for the Port of Havana Docks Company, two large deep-water piers, built entirely of reinforced concrete. These are the San Francisco and Machina Piers, two- and four-story reinforced concrete structures with 85-ft. concrete pile foundations, in deep water and soft mud. Two additional piers are planned to be built at some future date.

The piers were designed and the construction directed by Barclay Parsons and Klapp, Consulting Engineers, of New York City. The contractors were MacArthur, Perks and Company, also of New York City.

Although these piers, with the large connecting warehouse along the shore, contain many features of interest to engineers, the present discussion will be confined to the pile foundation.

The piers are 164 ft. wide and from 660 to 680 ft. long, the distance between them being 262½ ft. The depth of the water varied from 20 to 40 ft., and the piles were driven into 15 to 20 ft. of soft mud which covered a firm sand and clay bottom.

Prior to deciding on the use of concrete piles, careful consideration was given to various other types of foundations, but these piles were finally selected because, of all the permanent types of foundations,

* Continued from September, 1914, *Proceedings*.

† New York City.

Mr. Douglas.

they were the cheapest. Marine borers are particularly destructive in these tropical waters, and this made it quite impossible to use wooden piles.

The length of the piles varied from 45 ft. inshore to 85 ft. in the outer 300 ft. The variation in length was made in 5-ft. increments. The piles were square in cross-section, the length of the sides being 16, 18, or 20 in. The piles were reinforced longitudinally with from 4 to 6-ft. plain round steel bars varying from $\frac{3}{4}$ to $1\frac{1}{4}$ in. in diameter. These main rods were hooped with $\frac{1}{4}$-in. square rods every 16 in. in the length of the pile, except where the rods of opposite faces lapped, and there the hoops were 4 in. from center to center. In the lower 3 ft. of each pile the hoops were 6 in. from center to center, and in the upper $3\frac{1}{2}$ ft. there was a special rod reinforcement, basket-like in appearance, in order to resist the shock of the hammer. The details of the reinforcement are shown on Fig. 1.

For handling the piles, the longitudinal reinforcement was proportioned to the bending-moment stresses when the pile was supported horizontally from two points which were located so as to make the negative moments at the points of support about equal to the positive moment at the middle. Two methods of handling the piles were predetermined when they were designed. Fig. 2 shows one of these methods; the pile is about to be lowered into position by the derrick boat. The stresses resulting from handling the piles in this manner, and without allowance for impact, were 20 000 lb. per sq. in. of steel. In a few sample piles, high unit stresses were tried, but it was found absolutely impractical to handle them where the stresses were greater than 25 000 lb., but it was perfectly safe where they were somewhat less. In handling some of the first piles, before the hoisting engineers had had experience, the whipping of the piles caused failure, but this trouble lasted only a very short time, the men learning very quickly how to handle them properly. Experiments with hollow piles of somewhat lighter weight were not successful, as they were broken either in the handling or the driving. The heaviest solid pile weighed about 18 tons.

The casting was done in a yard especially equipped for the purpose. A level platform was first prepared on which the piles were cast, each being separated from the adjacent pile by a thin collapsible form. The piles were then constructed in layers, the second layer not being placed until the first one was reasonably hard. Each pile was numbered, and it was not touched for at least 3 weeks after it was cast. When sufficiently hard the piles were lifted and transported by overhead, electrically-operated, traveling cranes. These cranes were intended to be used on the piers after all the pile work had been completed. The pile casting area was roofed over in order to protect the laborers and the product from the sun.

Mr. Douglas.

TYPICAL DETAILS OF HOOPING ETC.

DETAILS OF CONCRETE PILES
FOR PIERS OF
PORT OF HAVANA DOCKS COMPANY
HAVANA, CUBA

FIG. 1.

Mr. Douglas.

Contrary to the usual custom, all the longitudinal rods were welded. The welding was done by Spanish blacksmiths, who were particularly efficient in such work. The reinforcement gave no trouble on account of the welds.

The piles, after hardening, were carried to the site of the work either on the derrick boat, Fig. 2, or on a lighter. They were lowered into place by the derrick, as shown on Fig. 3. The fall line was released quickly and the piles sank into the soft mud under their own weight, often penetrating it to as great a depth as 20 ft.

The pile was held by a sling fastened to pins which extended through two holes in the pile, provided in casting it, for the purposes of handling. Each hole was lined with a steel pipe in order to prevent injury in handling. After the pile had been dropped into place, the sling was released, and a 6-ton, No. 1, Vulcan, steam hammer, Fig. 4, suspended from the derrick boom, was placed on top of it. A rope mat was used to cushion the hammer blows, and each pile was driven with an average of 500 blows into a hard stratum until it gave a refusal of about ½ in. under the last ten blows. Occasionally, satisfactory refusal was not obtained, and, in such cases an additional pile was driven alongside to insure proper bearing capacity. There were 4 000 piles driven in this manner; they were arranged in clusters varying from 4 to 24. A number of piles were tested. Up to 60 tons there was very small settlement, the piles practically recovering when the test loads were removed. Each pile has a theoretic load of 40 tons, consisting of approximately 15 tons of dead load and 25 tons of live load. The tops of piles that could not be driven to the depths planned were cut off with a pneumatic chisel. Some trouble was caused by the concrete spawling off at the corners under driving. This was remedied by introducing ¼-in. square diagonal steel ties about 12 in. from center to center.

It was also found that calcarious sand gave very much stronger pile heads than ordinary sand or stone dust. The stone dust was particularly unsatisfactory, resulting in a great many breakages of the pile heads under driving.

Piles were driven at an average rate of about 12 a day. Only two lines of batter piles at the outer end of each pier were used, notwithstanding the great depth of water. For the lateral stiffness of the piers, the designer depended almost entirely on the resistance of the floor as a cantilever anchored in shore. This assumption seems to be warranted fully by the stiffness of the piers. As the concrete floors are heavily reinforced, it was assumed that any shock of boats against the pier would be transferred from the outer end, through the concrete and steel reinforcement, to a point where the piles were sufficiently short to insure ample stiffness. Fig. 4 shows the method of driving the batter piles.

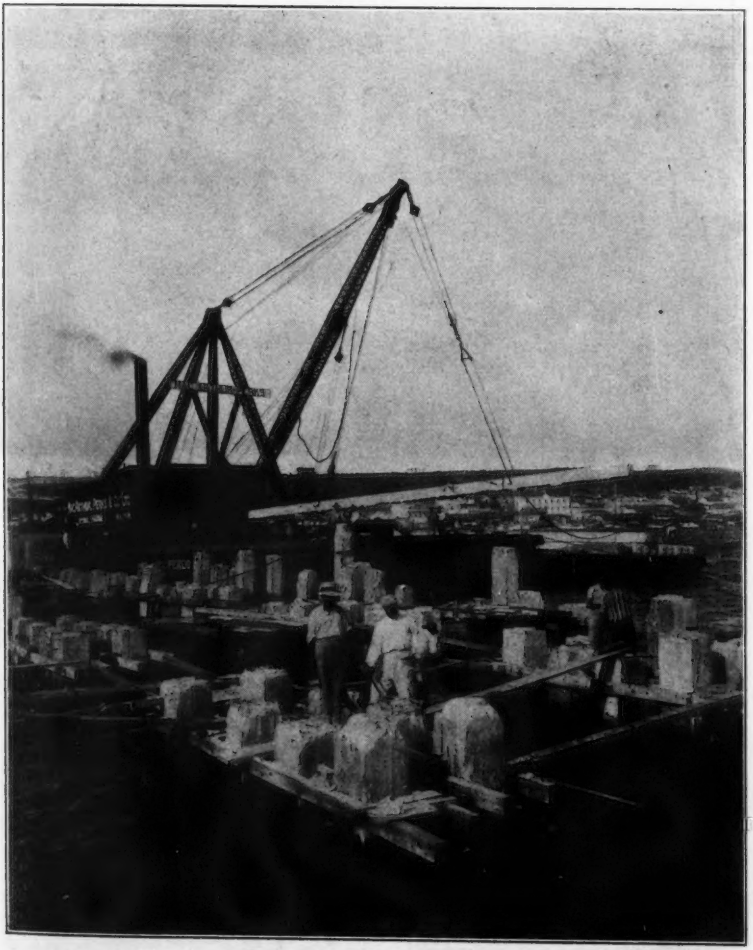


FIG. 2.—METHOD OF HANDLING THE PILES.

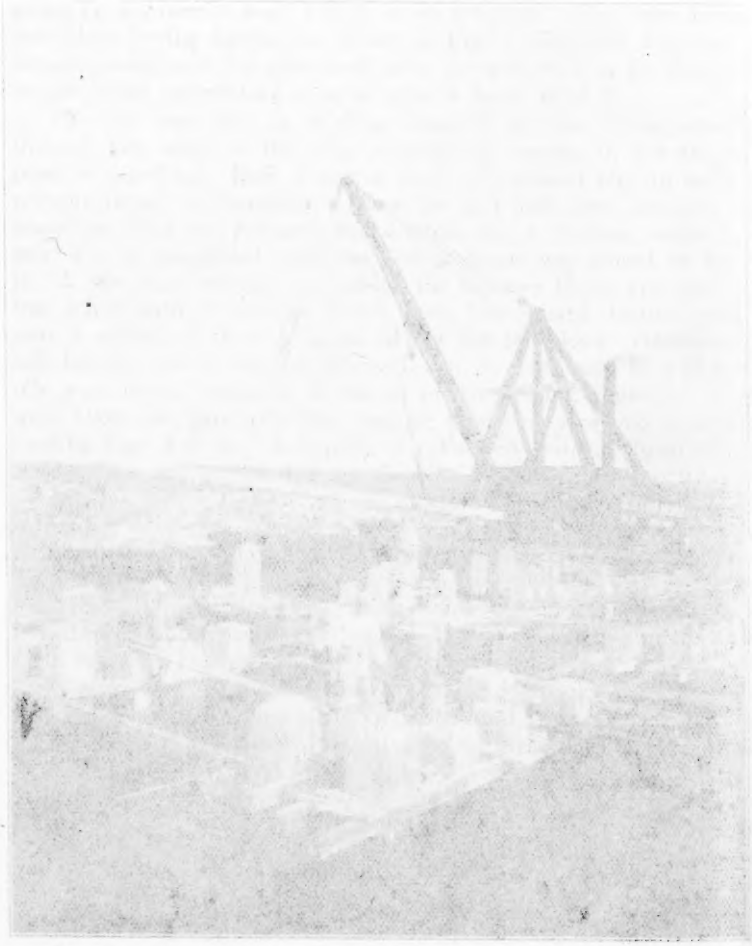


FIG. 3.—LOWERING A PILE INTO PLACE WITH THE DERRICK.

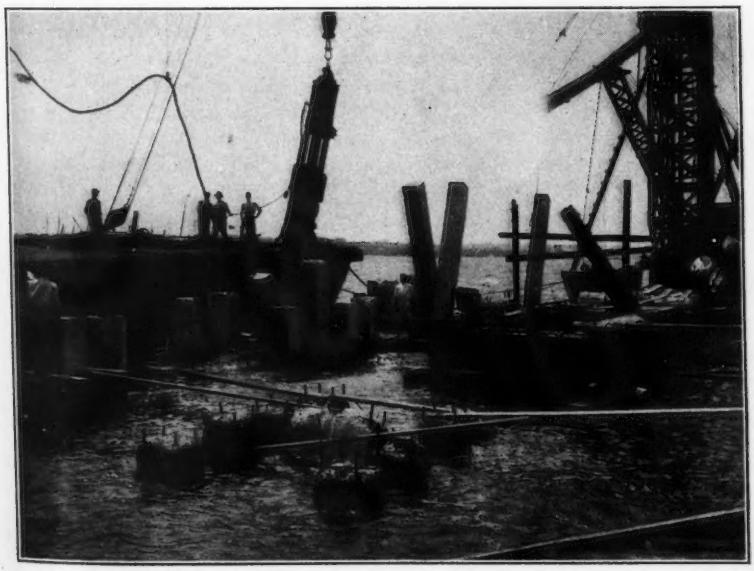
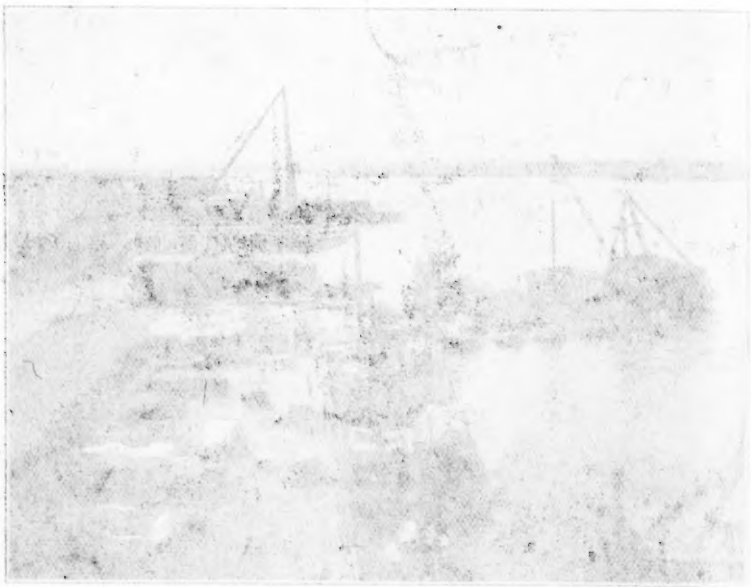


FIG. 4.—METHOD OF DRIVING THE BATTER PILES.



Mr. Douglas.

Previous to designing and constructing these piers, an examination was made of some concrete wharves in which the longitudinal reinforcement above water had caused the steel to rust and spoil the concrete. Quite a number of rods were exposed to sea action. This was due to the fact that the steel had not been spaced carefully. Extreme care was taken in this Havana work to keep the longitudinal steel well covered with concrete. In other cases it had been observed that, where tie wires had been left protruding from the faces of the piles, rusting had taken place and, of course, had worked in until there was disintegration of the concrete. The piles in these piers have been examined from time to time and have been found to be in perfectly good condition. At present, of course, they are fully covered with marine growth below the water line.

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PAPERS AND DISCUSSIONS

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SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

Discussion.*

By George P. Decker, Esq.

Mr. Decker.

George P. Decker,† Esq. (by letter).—The civil engineer has addressed himself in earnest to the water law problem. There are signs of it on all sides. Lawyer and layman must welcome the fact. The uses to which surface waters in natural bodies are put, has distinguished water from upland radically in connection with private rights. Water may be the subject of certain exclusive individual uses, but not more so than air. The uses of water, in most cases, are enjoyed in common by all members of the community, as are the uses of the air. Want of fixity of water distinguishes its character from that of land. The latter may be subdivided and the subdivisions may be enjoyed exclusively and harmoniously by individuals, but the former may not. These differences have resulted generally in recognition, sooner or later, of a necessity for governmental administration over water uses.

In newly settled countries, such as our States were recently, this administration begins with simple rules, and these increase in complexity as need for all the various water uses increases. Differences in natural surroundings lead to a different order in the development of such rules. In arid regions rules applicable to irrigation may be first developed, or may be the only ones developed. Along tidal waters rules are needed as to the use of riparian frontage in commerce much more than on interior waters. On power streams rules of a different sort, when developed at all, are found, and these are adjusted to those for other uses, according to the order or extent of the development of

* Continued from November, 1914, *Proceedings*.
† Rochester, N. Y.

Mr. Decker.

local industries. The common right of fishing is world-old; yet we have had State judges who have denied that it existed as to fresh waters. The denial of that common right was quite necessary in the effort to sustain the argument for denying the public right to water-power uses. Rules for the regulation of navigation, a common right, were developed at an early date, and so was active governmental administration over that right, because, in early days, of public dependence on streams for travel; the small stream as well as the large one. At first, the private user of water power was an outlaw; the power use might be enjoyed only where, and so long as, it did not interfere with boating—then the more important use. The power use, however, was soon legalized in all the States by express statutes, in order to protect the private user from prosecution in the name of the public, for his use was not at all in common with that of other members of the community. His use was exclusive, and, because of the necessary interference with the stream, was hostile to community uses for boating and also for fishing. The legalizing of that use by him was merely a temporary permission, a franchise, vouchsafed by the public. As long, however, as there was power enough, generally speaking, for every one, anything like real governmental supervision over it was in abeyance. In the United States that condition has lasted from the beginning almost to date, as to all active governmental supervision over public uses, municipal uses so-called and navigation being general exceptions, and irrigation a special exception in arid States.

Absence of supervision invited wide-spread litigation between conflicting private interests, and from that grew up a body of judge-made laws as to water uses, which came to constitute our water law so-called, but the law as recognized and widely applied. Mr. Johnston's criticisms of the quality of that law are admirably put and wholly justified. The only water uses in the Eastern States, which have demanded governmental supervision, so that these uses might be equitably enjoyed, were those of navigation and for municipal purposes. In applying that administration, however, the public was confronted with strange doctrines developed in the judicial decisions which had accumulated. It found that the exclusive use of a stream for power, enjoyed by the miller, had been recognized by the Courts as having a foundation in private property, as a vested right, not only as against his private neighbor, but as against all the rest of the public represented in its sovereignty by Government. That doctrine has been repudiated very recently by the United States Supreme Court in the Chandler-Dunbar Company case, arising at Sault Ste. Marie, and at intervals in earlier cases, some of them in the Courts of various States.

The enormous population with which we are now confronted, presenting itself almost everywhere, makes its own demand for a common

Mr. Decker.

enjoyment of the naturally public uses of water and for the enforcement of the public right thereto. We face a necessity, therefore, for active governmental administration of all such uses, for without it any real public enjoyment is quite out of the question. The rights of the public, of course, will be respected. That they should be disregarded for long, after enjoyment of them has become essential to public welfare, is impossible to conceive. No judge can rule irrepealably that these uses shall, for all time, at this place or at that, constitute private property in the few owners of adjoining upland. A new judge can be chosen who will overrule him. The development of the policies to be followed and the methods of administration of the various uses to which the public is entitled in bringing the common right or benefit to all, are among the most important tasks confronting the American people. The Engineering Profession, of course, must play its part. It is, in fact, the only profession qualified to-day to undertake that task. The department of the law which the legal profession has most ignored is that which deals with waters. No student entering that profession, up to the present, has had facilities for specializing in the law of waters, if he had desired. In the course offered by law schools, the law of waters is not usually dignified with provision for special instruction. Few practitioners have been in such circumstances that they could devote themselves specially to this subject. The water cases which have come before the Courts have been taken there by lawyers who in almost every instance were engaged in their first and last water case. The judge who decided a water case and who was sure to write an opinion, possibly—if not probably—was deciding his first and last water case. The result has been that there is no group of decisions so devoid of uniformity in the application of principles, so contradictory in the rules applied, as those which have gone to make up our water laws. No State has yet developed an adequate code of water laws. Such provisions as exist are usually found scattered through statute laws on other subjects, partly in the highway law, partly in the fish and game law, and partly in the municipal law. Only a civil engineer could have enjoyed a training and experience which would lead to the broad view of the subject revealed by Mr. Johnston's paper.

Mr. Johnston declares, as a first principle, the "public ownership of streams, lakes, and other bodies of surface water." That statement is useful at this time for the purpose of pointing out the common and great error involved in the statement that there is a qualified or almost unqualified private ownership. The use of the term "ownership" in connection with the subject of waters is always unfortunate, for it leads to misconceptions as to rights. It has been the layman, however, who was responsible for introducing this term in this connection, and he demands perhaps that it be used.

Mr. Decker.

It is desirable in the early stages of the effort to introduce a sane and just system of water laws, in order to eliminate all error possible to be implied in the statement of principles. The writer is of the opinion that Mr. Johnston's statement embraces such an element. Although water uses include many which are naturally to be enjoyed in common by all members of the community, there is one at least which is not naturally shared in this way under our system of private ownership of upland. The private owner of upland adjoining a stream is, quite independently of any act of his or of interference with a stream by him, served with those benefits which come through the saturation of his land by the stream. Such a benefit may justly be termed a natural one. It is one which the earliest agricultural settler relied on in securing his location. On that saturation, certain crops may depend. Riparian lots available for dwelling purposes may also have a value, due to the natural beauty of the contiguous stream. If so, that value was placed there by Nature. The stream may have a flow sufficient to be serviceable as a highway for boating, and, if it has, it was rendered so (opened) by Nature and not, as a rule, by Government, and is an outlet for the riparian land owner. In the case of highway openings by Government and the consequent invitation to settle and improve locations adjoining it, on the assumption that the highway will be perpetuated, it is recognized as just, if not indispensable, in closing the highway, or in shifting it, or in making successive changes in street grade, that compensation should be paid the abutting owner according to the injury which may result.

Diversion of a stream, then, without recognizing any obligation of the public to pay any damages—diversion on the theory that the public owns the stream (may destroy it to the extent of diversion) and render no compensation to any one—may involve injustice. This view is not based on recognition of any right in a private abutting land owner to interfere artificially with a stream, to divert, pollute, or obstruct it. The benefits referred to are such as are natural. If stream flow is to be diverted, then, by the public, in connection with an intended improvement facilitating public enjoyment of a common use or uses, or if a fraction of the flow is to be diverted, just damages according to the injury done should be paid. Such cases would be unlike that of the removal of a public building, artificial in itself, and where the gratuitous benefits which had come to neighboring lands by its construction would be lost on its removal. If taken away without compensation, there has been no injustice inflicted by the public. The law as administered has usually failed in the proper grouping of the cases which may afford a just ground for private damages against the public in case of complete or partial diversion of stream flow. If, however, damages are to be awarded when justly called for, as here

indicated, no serious obstacle in the way of the proper development of streams for their highest public usefulness would be presented. Mr. Decker.

The deterring of public improvements and of desirable governmental supervision over streams, has been due to the gross injustice to the public arising so often when damages have been awarded on the mistaken theory that uses which were in principle public and common were exclusively private, and private in perpetuity. The commonest example of that has been in case of power uses, for those, when treated as based on rights of private property, exclusive and perpetual, and then capitalized on the basis of their economic value, lead to startling figures in the making up of Court awards. If a private property right, as against the public, to use the water power of a stream for private purposes when the public needs it for lockages, cannot be sustained in justice, the same reasoning should apply where the public needs to divert the stream for municipal supply or for public power purposes. The stream in any such case would be taken for public purposes. It is for the Government, through its legislative department, to say which of all public uses shall have the right of way over the others. The riparian owner, from whose private frontage a stream is diverted, then should be paid no element of damages on the theory that as such land owner he has any exclusive right of use of the stream at his front for boating, for fishing, for drinking, or for artificial diversion for ends of his own. His damages in case of diversion by the public, when confined to loss of natural benefits, would consist of a nominal award in most cases, and a modest award in any case—so small in most cases that he would not seek an award.

A statement of principles based on recognition of the public right may be formulated as follows: Water uses which the public may enjoy in common are inalienable by Government for private benefit; they are subject at all times to regulation and administration by Government; if the public diverts a stream, it is liable for the injury, if any, produced by cutting off the natural irrigation or other natural benefits received from the stream by private riparian lands; if the channel below the diversion was navigable, the public is also liable for the consequent depreciation, if any, in the value of the lands fronting thereon; if the public is to acquire the right to divert forever, the damages, of course, will be estimated accordingly.

We seem already to have reached the time when denial, tantamount to betrayal, by State Courts of the public rights in respect to water uses must cease. Two regrettable instances of that have occurred recently, one in the Court of Appeals of New York, in the Fulton Light, Heat and Power Company case, the other in the Supreme Court of Wisconsin, in the Wausau Street Railroad Company case. In both these cases, the Courts applied the principle requiring protection of private property provided for in Federal and State Constitutions to

Mr. Decker.

safeguard it from unjust attacks by the public. It is now necessary that all concerned shall recognize the principle of greater sanctity and importance, namely, that public rights shall be safeguarded against attack by private interests. This principle is so fundamental the world over that constitution makers have not deemed it necessary to affirm it specially. It applies in the case of very many fundamental rights, of which no mention is made in written constitutions. It is gratifying to know that the highest judicial tribunal in the United States respected and enforced this principle in the case of a water use subject to public enjoyment, in the decision reached in the Sault Ste. Marie case in 1913 (229 U. S., 53), where a lower Court award of $500 000 against the public, in favor of a private corporation claiming a monopoly in the use of that river for private enjoyment, was reversed.

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PAPERS AND DISCUSSIONS

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THE CONSTANT-ANGLE ARCH DAM.

Discussion.*

By Orrin L. Brodie, M. Am. Soc. C. E.

Mr. Brodie.

Orrin L. Brodie,† M. Am. Soc. C. E. (by letter).—The author is to be congratulated on the generally clear and concise treatment of the subject matter of his valuable paper and on his demonstration of the constant-angle arch principle. The paper has proved of particular interest to the writer as he has had occasion several times in the last few years to analyze the stresses in arched dams of different types. He gladly greeted the author's statement, early in his paper, of his presentation of a short-cut method for finding the distribution of the total load between arch and cantilever elements of the structure. The writer had developed a set of general formulas (including the moment of inertia of the horizontal section of the cantilever portion considered between vertical radial planes), so that a direct solution was possible for a dam of any vertical cross-section; but the actual computations prove long and tedious. The formulas referred to were developed along the lines laid down by the late R. Shirreffs, M. Am. Soc. C. E. In connection with this approximate short-cut method the writer, not having opportunity to try it for himself, would like to know whether it is applicable to a dam of any cross-section, say a spillway dam (at least 120 ft. high), arched, and which requires many changes, particularly at the top and bottom, in slopes of its down-stream face. He would also like to know what expenditure of time would be required in determining completely, by the ingenious short-cut method of the author, and his distribution for a dam of the indicated type.

The author mentions a typographical error in the original of the arch deflection formula of the late Mr. Shirreffs, relating to a trigonometric function of θ. The derivation of this formula, it is true, will result in the expression as written by the author in connection with

* Continued from October, 1914, *Proceedings*.

† New York City.

Mr. Brodie.

Fig. 6, but it was originally written, evidently, to retain the angle, θ, throughout, instead of involving 2 θ, as the author has written in the numerator of the first term of the expression for CC_c. That is, the last term of this numerator, $\frac{1}{2}$ (cos. 2 θ — 1) is equal to (cos. $^2\theta$ — 1), as originally expressed.

The author's Formulas (6) and (7) are open to some question, in the writer's mind.

The intensity of stress due to the weight of a large dam, reservoir empty, is not generally uniformly distributed in an up-and-down-stream direction, over the foundation, though for a thinner arch ring this uniformity of distribution may be approached. Furthermore, the initial axial stress (in terms of head of water) due to the weight of the dam, assuming for the time that its weight is uniformly distributed over the base or foundation, will, with Poisson's ratios taken at $\frac{1}{4}$, be as the author states, $\frac{1}{3} \times 1.27\ H$, where H is the height of the dam above the horizontal layer at the foundation level. Unyielding abutments are supposed to resist this axial strain; but the resulting strain in the horizontal direction at right angles to the axial strain, up and down stream, is opposed by the water pressure only on the up-stream side of the dam at the given level. Yet, with the water level at the top of the dam, a head of $\frac{1}{3}\ H$ is assumed to oppose this up-and-down-stream strain. The unrestrained down-stream side has been neglected in considering $\frac{1}{3}\ H$ as in Formula (6), by the author.

In other words, unless there be a force on the down-stream side, opposing the water pressure on the up-stream side, it is not clear to the writer how the initial axial stresses can be considered to resist the intensity of pressure due to a head of water, h, nor is it at all clear how this "resistance" can be so definitely determined, as stated in Formulas (6) and (7), with the above mentioned facts in mind.

In closing, the writer wishes to emphasize the thought expressed by the author that there is no single type of dam that may readily be applied to every site. Each site provides its own problem to be thoroughly investigated.

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A METHOD OF DETERMINING STORM-WATER RUN-OFF.

Discussion.*

By Messrs. Samuel D. Bleich, Charles E. Gregory, and Samuel A. Greeley.

Mr. Bleich.

Samuel D. Bleich,† Esq. (by letter).—Mr. Buerger's new run-off formula again calls attention to the uncertainties and general confusion surrounding the technique for determining the run-off from the valuation of the numerous elements on which it depends. Almost every conceivable physical feature of a drainage area affects the run-off in some manner, but the relation of each can only be described qualitatively. These features have not as yet been adequately expressed mathematically.

Mr. Buerger gives a résumé of the various empirical formulas which have been proposed for the computation of run-off in terms of the salient characteristics of the drainage areas, such as size, general slope, relative imperviousness, and the length of the sewer. These formulas, it is claimed, are defective and give inconsistent results, as they were devised to fit local conditions. His objection to the so-called rational or "time element" method is that it is difficult of application; and he proposes his formula as its "acceptable equivalent."

The expression which Mr. Buerger gives for the run-off is necessarily complicated, as the rate of run-off per acre is made to depend implicitly on six elements or characteristics of the drainage area. In other empirical formulas, such as those by McMath, Bürkli-Ziegler, and others, the run-off is given explicitly in a simple relation, in terms of four elements only: area, slope, imperviousness of ground, and rainfall intensity. As a result of the complicated form of the proposed

* Continued from November, 1914, *Proceedings*.

† Asst. Div. Engr., in Chg. of Sewer Design, New York Public Service Commission, First Dist., New York City.

Mr. Bleich.

formula, three diagrams are required in solving it, whereas, for the other formulas, one is sufficient. It would be interesting to know how the form of the new formula was devised.

The rational method, as developed in actual practice, although not as simple as the standard empirical formulas, is neither more difficult nor much more time-consuming than the proposed formula. It appeals more to reason and, on account of its simplicity and directness, meets with general favor. It is true that it has its shortcomings, but so have all the empirical formulas, and probably to a greater degree. The empirical formula is involved, and the relations of the factors are obscure; in consequence, it is not expected that it will come into extensive use. Its advantage over the simple empirical formulas is not evident and, in addition, it involves certain inconsistencies which destroy its validity.

It is believed that Table 20 and Fig. 7 will facilitate the use of the Buerger formula. Table 20 contains computations of the values of N and P, and Fig. 7 solves the expression,

$$q + q^{\frac{4}{5}} N = P,$$

by a series of straight lines. With N and P obtained from Table 20, q may be obtained from the diagram, Fig. 7, at the intersection of these values for N and P.

It is generally conceded that the larger the drainage area, other conditions being equal, the smaller will be the run-off per unit of area. This is contradicted by Mr. Buerger's formula. Examining the expression for N it is seen that N decreases as A increases; and, from the expression for q, q increases as N decreases; therefore, q increases as A increases. It is conceivable that two slightly different areas, may have the same values for m, K, S, C, and L, and for these two areas Mr. Buerger's formula will give a larger run-off per acre for the larger area, which is contrary to experience. The increase of run-off per acre with increasing areas is not large, but being in the wrong direction, it militates against the validity of the proposed formula.

Mr. Buerger's formula, therefore, appears to be contrary to reason and experience in the relation which it involves between q and A; and in this respect it certainly has no advantage over the other empirical formulas which he mentions. McMath's formula is approximately consistent, and, by a proper selection of values for the two constants, C and R, it may be made to give satisfactory results. It certainly is easier to apply than the new formula.

In practice, the writer prefers the use of the rational or "time element" method, the results to be checked systematically by McMath's formula, with values for the constants suitably selected in both cases.

Mr. Bleich.

DIAGRAM FOR THE SOLUTION OF THE
PROPOSED RUN-OFF FORMULA BY MR. C.B. BUERGER.

$$q+q^{\frac{4}{5}}N=P;\quad N=\frac{L}{79\sqrt[5]{CAS^2}\ (24+\frac{16}{\sqrt{m}})};\quad P=\frac{3.33\sqrt{m}KC}{\sqrt{m}+0.87}.$$

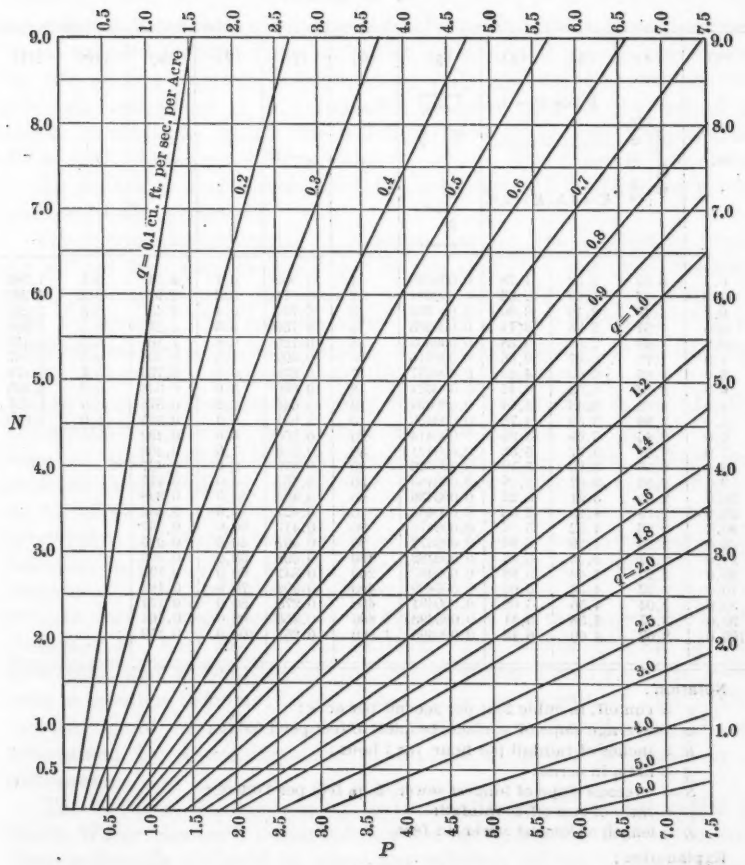

FIG. 7.

Mr. Bleich.

TABLE 20.—COMPUTATION OF VALUES OF N AND P IN THE RUN-OFF FORMULA PROPOSED BY MR. BUERGER.

$$q + q^{\frac{4}{5}} N = P; \quad N = \frac{L}{79 \sqrt[5]{C A S^2} \left(24 + \frac{16}{\sqrt{m}}\right)};$$

$$P = \frac{3.33 \sqrt{m} K C}{\sqrt{m} + 0.87}.$$

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
		P for $C = 1$.								
m.	$\frac{3.33\sqrt{m}}{\sqrt{m} + 0.87}$	$K = 1.5$.	$K = 2.0$.	$\left[79\left(24 + \frac{16}{\sqrt{m}}\right)\right]^{-1}$	A.	$A^{-\frac{1}{5}}$.	S.	$S^{-\frac{2}{5}}$.	C.	$C^{-\frac{1}{5}}$.
0.1	0.89	1.33	1.78	0.000168	1	1.000	0.1	2.51	0.1	1.585
0.2	1.13	1.69	2.26	0.000211	2	0.871	0.2	1.91	0.2	1.380
0.4	1.40	2.10	2.80	0.000256	3	0.794	0.4	1.45	0.3	1.274
0.6	1.57	2.35	3.14	0.000283	4	0.759	0.6	1.23	0.4	1.202
0.8	1.69	2.53	3.38	0.000303	5	0.725	0.8	1.10	0.5	1.151
1.0	1.78	2.67	3.56	0.000316	6	0.692	1.0	1.00	0.6	1.107
2.0	2.06	3.09	4.12	0.000357	7	0.676	2.0	0.759	0.7	1.074
3.0	2.22	3.33	4.44	0.000384	8	0.660	3.0	0.646	0.8	1.045
4.0	2.32	3.48	4.64	0.000394	9	0.646	4.0	0.576	0.9	1.021
5.0	2.39	3.59	4.78	0.000406	10	0.631	5.0	0.525	1.0	1.000
6.0	2.46	3.69	4.92	0.000414	20	0.550	6.0	0.490		
7.0	2.51	3.77	5.02	0.000421	30	0.513	7.0	0.468		
8.0	2.55	3.83	5.10	0.000426	40	0.479	8.0	0.437		
9.0	2.58	3.87	5.16	0.000433	50	0.457	9.0	0.417		
10.0	2.61	3.91	5.22	0.000436	60	0.437	10.0	0.398		
20.0	2.79	4.19	5.58	0.000458	70	0.427	20.0	0.302		
30.0	2.88	4.32	5.76	0.000470	80	0.417	30.0	0.257		
40.0	2.93	4.39	5.86	0.000477	90	0.407	40.0	0.229		
50.0	2.97	4.45	5.94	0.000482	100	0.398	50.0	0.209		
60.0	2.99	4.49	5.98	0.000487	200	0.347	60.0	0.195		
70.0	3.02	4.53	6.04	0.000489	400	0.302	70.0	0.182		
80.0	3.04	4.56	6.08	0.000491	600	0.275	80.0	0.174		
90.0	3.05	4.58	6.11	0.000493	800	0.263	90.0	0.166		
100.0	3.07	4.60	6.14	0.000495	1 000	0.251	100.0	0.159		

Notation:

q = run-off, in cubic feet per second per acre;
m = average slope of surface, to inlet, in feet per 1 000 ft.;
K = inches of rainfall per hour, for 1 hour;
A = area, in acres;
S = average slope of longest sewer, L, in feet per 1 000 ft.;
C = ratio of run-off to rainfall;
L = length of longest sewer, in feet.

Explanation:

P is obtained by multiplying Column 2 by $K\,C$.
Columns 3 and 4 give P for $K = 1.5$ and $K = 2$, for $C = 1$.
N = Column 5 × Column 7 × Column 9 × Column 11 × L.
With N and P known, q may be computed, or obtained from Fig. 7.

Mr. Gregory.

CHARLES E. GREGORY,* ASSOC. M. AM. SOC. C. E.—The author has not only felt that storm-water run-off formulas have been popular, but he has also felt, as have many others, that the results obtained have not always been reliable, and his aim has been to devise a formula which would be as reliable and as susceptible to adaptation to varying conditions as the so-called rational method.

In order to add to the reliability and adaptability of the formula presented, the author has introduced more factors than have been commonly used, and, in this case, it has resulted in a rather involved and difficult expression. The speaker believes that the rational method should be used for all important sewer designs, and does not advocate the use of any formula except where only approximate results are required, and where it is permissible to save time by the use of a simple formula in which the relative importance and significance of its coefficients are understood by the user. The formula presented by the author is not simple, and the relations of its parts have not been rationally explained.

The exponential formulas in common use are the simplest in form, but do not account for variation in the shape of the area drained, or the arrangement of the drains, and do not allow for variation in time for entry. The possible error in using these formulas, due to variation in shape, ranges from 15% in one direction to about 25% in the other, when compared with the average watershed.

The formula presented by Mr. Buerger contains a factor, L, for the greatest length of drain, and uses the factor, m, to indicate a slope of land surface to the sewer inlets. The other factors are included in the ordinary exponential formulas. The use of the factor, m, to allow for variation in the time for the water to reach the sewer inlets is not sufficient, as the slope is by no means the controlling influence in the time of entry. The length of run is of quite as much importance, and, in built-up sections, the type of roof collection, as brought out by the speaker in a discussion† of the paper by C. E. Grunsky, M. Am. Soc. C. E., entitled "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem", will have a greater influence than either slope or length of run. This variable could be represented much better by a simple time factor ascertained by a consideration of all the elements by which it is influenced.

The speaker, in his paper‡ entitled "Rainfall and Run-Off in Storm-Water Sewers", presented two formulas, one in the exponential form rationally deduced to show the relation of the various factors to each other and make possible a reasonable modification of the con-

* Mt. Kisco, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 390.

‡ *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 458.

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stants, and one for use in connection with the rational method to get a first approximation. Under average conditions, this formula gives accurate results, as every factor in it may be directly modified to fit exactly all the assumptions which might be made in the rational method, except the expression for average velocity, which was necessarily based on average conditions, and is expressed in terms of area and average slope. The value of this average velocity, however, can be very easily and quickly tested and corrected, as soon as the first approximate result has been obtained, by using a Crane slide-rule, as modified by the speaker to give velocities directly. The basis of this formula is the ordinary rainfall intensity curve in the form:

$$I = \frac{B}{t + D}$$

in which I = the rate of rainfall, in inches per hour, B and D are empirical constants, and t = the duration of downfall, in minutes. In the original form used $B = 105$, $D = 15$, and $I = \frac{105}{t + 15}$. By changing the constants, B and D, this curve can be made to fit almost any rainfall curve desired. Assume that $t = \frac{L}{V}$, that L is the greatest length of drain, and V is the average velocity of flow; express V in terms of A and S, add to D a time for entry, in minutes; apply a coefficient, C; assume that inches per hour of rain are equivalent to cubic feet per second per acre, Q, and the rainfall curve becomes a rational run-off formula with all its elements easily understood and in a much simpler form than the one presented. If it is desired to use the intensity curve for New York City which gives intensities which will probably occur not oftener than once in 10 years, on an average, B will = 150 and D will = 15; allow 5 min. for the rain to reach the inlets, and the entire run-off formula for such assumptions will be as follows:

$$Q = C \frac{150}{\left[\frac{L}{84 \sqrt[5]{A S^2}} + 5\right] + 15}.$$

The coefficient, C, representing the ratio of rate of run-off to rainfall, in this or any of the exponential formulas, may be assumed to be influenced most by the percentage of imperviousness, as is done by Mr. Buerger where he says the theoretical range is from 0 to 1.00, corresponding with the imperviousness of the soil, but such an assumption will not give results even approximately in agreement with gaugings. His assumptions for what he calls "less unusual conditions" ranging from 5 to 50%, as a function of population, are more nearly correct for rain periods not longer than the time for concentration.

Mr. Gregory.

This factor, C, is not, as stated by the author, the ratio between the run-off and total rainfall, but is the ratio between the maximum rate of run-off and the average rate of rainfall for the time of concentration, and is governed by a combination of influences which may be shown in their relation by the following formula:

$$C = e\,(f\,g + (1 - f)\,h\,g) = e\,g\,(f\,(1 - h) + h);$$

e = coefficient of distribution which is a function of area;
f = percentage of impervious area;
g = ratio of run-off to rainfall due to storage on catchment surfaces and in the drainage structures; it is a function of t, and is influenced by the time for concentration and the relative area and character of the time zones;
h = ratio of run-off to rainfall due to perviousness of soil; it is a function of t and I.

The speaker, in a previous paper, deduced empirical values for C in terms of t. The value for C, as deduced from gaugings in the Sixth Avenue sewer in New York City, may be obtained by assigning values in the above formula as follows:

$$e = 0.9,\ f = 0.9,\ g = 0.5, \text{ and } h = 0.10;$$

then
$$C = 0.9 \times 0.5\ (0.9\ (1 - 0.10) + 0.10) = 0.41.$$

Additional data are being collected by the speaker, the better to establish values for e, g, and h. The factors, e and g, vary in opposite directions, tending to balance each other, and, for ordinary and similar conditions, a constant value for C may be used for maximum results without very serious error, after it has been established by giving due weight to its various factors. Assumed relations between population and impervious area should be used with great care, and should not be expected to give accurate results.

The use of diagrams to give results for various factors of a formula is to be commended, and the values of e, g, and h can be expressed much better in this way, as the law of variation for each changes at various points.

Studies have been made by the speaker to show the relation between the size of the sewer and the amount of damage done by flooding at various time intervals. Curves showing the relation between cost of sewers of various capacities, expressed in terms of frequency of flooding, to corresponding damage costs indicate that for certain cheap developments it would be more economical to have no sewer at all; though, at the other extreme of city development, similar to parts of the Borough of Manhattan, New York City, one flood would cost far more in damages than a sewer large enough to take the maximum storm. An important factor of sanitation is convenience and

Mr. Gregory.

suitability, which must be counted in favor of a sewer, or a larger sewer, and on this it is difficult to place a financial value. It is not wholly a problem of finance.

The curve given for New York on the diagram of maximum rates for 1 hour, Plate XIII, does not agree with data for the continuous 45-year record at the Arsenal in Central Park. This record gives values almost identical with the St. Louis curve shown on the same diagram.

In his discussion, Mr. Sherman gives Manhattan, New York City, as a place where the Hering formula is used. This is no longer true, as the rational method now prevails in that Borough.

Mr. Greeley.

SAMUEL A. GREELEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read with much interest this latest contribution to the important subject of storm-water run-off. The paper shows evidence of a large amount of work, as most of the available sewer gaugings are tabulated and checked by Mr. Buerger's formula.

The author has made a very fair statement of the various formula methods and the rational method for computing storm-water run-off in accordance with present practice, and it is his conclusion that the rational method has not received the widest use because it is laborious and requires a material exercise of judgment. This would indicate that two elements of value in the formula method are simplicity and speed of computation, and the elimination of the use of judgment. The writer agrees that the feature of chief value in that method is simplicity. He believes, however, that the exercise of judgment in the design of storm-water sewers should be promoted rather than discouraged. Even a formula computation will not give satisfactory results for any one locality, unless the user's judgment has been trained by experience in the use of the particular formula under existing local conditions. The Chicago run-off formulas computed by Mr. Hill,† are among the simplest in use and, with proper diagrams, permit of rapid computation. It appears to the writer that, in simplicity and rapidity, the proposed formula and the diagrams for its solution do not compare with some others now in use. It must stand, therefore, on its wider and more accurate application.

The formula proposed by Mr. Buerger, as shown in Equations 9, 10, and 11, is not a simple expression, and is not readily comprehended by engineers who have not had the opportunity to make special studies of formula derivation. The following comments, with reference to the units used in the formula, are submitted.

Coefficient, C.—It is not clear, from the definition given by the author, whether C is a ratio of the total volume of rainfall eventually reaching the sewers to the total rainfall, or the ratio of the maxi-

* Winnetka, Ill.

† *Journal*, Western Soc. of Engineers, October, 1902.

Mr. Greeley.

mum rate of run-off to the average rate of rainfall during the period of concentration to the sewer under consideration. The values of C, as determined in actual gaugings, represent the latter ratio.

Slope, S.—The author defines the factor, S, as the average slope of the longest sewer, L, in feet per 1 000 ft. It is not clear whether this means the composite average of a series of sewer lines at different slopes or the average slope of the sewer between two points, irrespective of breaks in the grade. The latter interpretation of this factor would simplify the computations.

Average Slope of Ground, m.—In his formula the author introduces a factor to represent the average slope of the ground surfaces to the sewer inlets. The writer's experience has been that this factor is difficult to ascertain accurately in practice. A large sewer area covering a broken surface will show considerable variation in the average slope at different points along the line. The relative weight of these different slopes in the general average is hard to determine. In the rational method this factor is accounted for in the time allowance for the flow of storm-water from roofs, yards, etc., to the sewer inlets. This time allowance is added to the time of concentration for the sewer, and is ordinarily not more than 15% of the total concentration period and is frequently less. Errors in the assumed time of entrance into the sewer are, therefore, of less relative importance.

Rainfall per Hour, K.—The author uses a rainfall factor equal to the expected rate, in inches per hour for a period of 1 hour, and recommends that sewers be designed to receive the run-off from rains of an intensity which may be exceeded once every year. The writer believes that this is too frequent a surcharge for many cases, and that in certain more critical conditions provision should be made for a higher rate of rainfall.

A comparison of the work required to determine the values of the factors entering into Mr. Buerger's formula with similar steps required in the rational method does not appear to the writer to favor the new formula. The coefficient in both cases must be selected with judgment. In the Buerger formula, it is necessary to determine the length of the longest sewer run and its average slope. With these factors determined, it is not difficult to estimate the average velocity and resulting time of flow in the sewers. The factor, m, required in the Buerger formula is, in the writer's judgment, difficult to determine with accuracy, and is not as easy to apply as the constant time factor used with the rational method for the flow time of storm-water to the sewer inlet. The rainfall factor, K, in the Buerger formula, will be constant for any one vicinity, and, therefore, the selection of varying rates of rainfall corresponding to different times of concentration is eliminated.

Mr. Greeley.

The writer has recently designed a system of storm-water sewers for a town in the West having a population of about 5 000. The town was divided into three storm-sewer districts. A ridge about 40 ft. high runs through these districts near the center, separating two flat areas, a lower and an upper. In one of these districts it was necessary, in order to avoid excessive cuts, to carry the main sewer around the end of the ridge. On this account the length of sewer run and the time of concentration for District B were considerably greater than those for District A, although the two districts were not very different in area. The writer made the computations for storm-water run-off in accordance with the rational method and also with Mr. Hill's modified Bürkli-Ziegler formula. The coefficient used with the rational method was determined from a series of gaugings made on the combined sewers now used in the town, in accordance with Table 21, so that a very fair determination of this factor was possible. Since making these computations, the writer has computed the storm-water run-off by the Buerger formula, and was interested to find that the results compared very favorably with the determinations according to the rational method. In both computations, the smaller run-off was estimated for the larger area on account of the longer time of concentration. The results of these computations are shown in Table 22. In thus using the Buerger formula, the writer found it difficult to make an accurate estimate of the value of m, therefore he made two computations, using different values of m.

TABLE 21.—GAUGINGS IN CHERRY STREET SEWER, WINNETKA, ILL.

Date, 1912.	Time before maximum discharge, in minutes.	Area tributary in this time, in acres.	Inches per hour.	Cubic feet per second.	Maximum flow in sewer in excess of dry-weather flow.	C.	Remarks.
July 20th.....	75	381	0.163	62.1	13.5	21.7	Sudden moderate shower.
July 20th.....	50	320	0.200	64.0	15.0	23.4	12 hours after previous storm.
July 23d.....	25	160	0.450	73.9	15.5	20.9	Short, sharp shower.
July 28th.....	60	365	0.154	56.2	12.3	21.8	Second of two showers.
August 8th.....	30	195	0.390	76.8	11.7	15.2	Short, quick storm.
August 9th.....	75	381	0.22	82.3	13.0	15.8	8 hours after previous storm.
November 12th..	20	130	0.90	116.8	20.2	17.3	Sharp, short shower.

There is one point about Mr. Buerger's paper which deserves comment. He has tabulated most of the recorded gaugings of storm flows in sewers, and has compared these gaugings with results computed by his formula method. It would appear that the formula method should give the maximum run-off to be expected from the local conditions of each gauging. The gaugings, however, do not show maximum run-offs, but, in many cases, figures which are considerably below

the maximum run-off. The comparison should be judged with this fact in mind. Mr. Greeley.

TABLE 22.—COMPUTATIONS FOR STORM-WATER RUN-OFF BY DIFFERENT METHODS.

Factors.	Buerger Formula.				Rational Method.		Hill's Formula.	
	District.		District.		District.		District.	
	A	B	A	B	A	B	A	B
A = Area, in acres	160	180	160	180	160	180	160	180
L = Length of longest sewer, in feet	5 600	9 100	5 600	9 100	5 600	9 100	.....	.....
C = A coefficient	0.20	0.20	0.20	0.20	0.20	0.20	0.55	0.55
K = Rate of rainfall, in inches per hour	0.9	0.9	0.9	0.9	1.2	0.95	1.0	1.0
S = Average slope of sewer, L, per thousand	7.8	5.2	7.8	5.2	7.8	5.2	.....	.....
m = Average slope of ground, per thousand	7.0	5.0	2.0	1.0	.....	.....	.....	.....
T = Time of concentration, in minutes	.....	.....	.....	.....	33	50	.....	.....
P	0.5	0.45	0.4	0.35	.....	.....	.....	.....
N	0.5	0.9	0.4	0.75	.....	.....	.....	.....
Q = Total run-off, in cubic feet per second	48.0	36.0	41.6	30.6	38.5	34.2	24.0	25.2

In conclusion, the writer finds that this formula gives satisfactorily accurate results as compared with the rational method for the particular application made, but he does not consider it easier to use, and he does not favor attempts to reduce the exercise of judgment in storm-sewer design. The value of m is not easily determined. It would seem to him that engineers should attempt to secure data and gaugings which could be used as guides for judgment, rather than to reduce the importance of the judgment factor. The author's discussion of run-off has added materially to the data on that subject, and his suggestions as to the frequency of surcharge and the general similarity of rainfall intensities over comparatively wide areas, have simplified the selection of rainfall rates in storm-sewer design, and thus have made easier the exercise of judgment along this line.

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PAPERS AND DISCUSSIONS

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THE DESIGN AND CONSTRUCTION OF FOUR REINFORCED CONCRETE VIADUCTS AT FORT WORTH, TEXAS.

Discussion.*

By Carl Gayler, M. Am. Soc. C. E.

Mr. Gayler.

Carl Gayler,† M. Am. Soc. C. E. (by letter).—It is not often that important structures such as these concrete viaducts are described as fully and clearly as in this paper. Publications of this kind are of great value to the Profession.

Reinforced concrete viaducts of the general layout and appearance of the four Fort Worth viaducts are not uncommon, but the construction of arches by using steel centers embedded in the concrete of the arches and forming an integral part of them is of recent date.

At first glance, comparison with the Melan arch suggests itself; there are, however, the following radical differences between the two systems:

1. The steel ribs of the Melan arch are not designed to be self-sustaining over the full length of the span under the weight of the concrete of the arch proper.

2. The usual assumption for the computation of Melan arches—that the concrete resists the direct thrust and the steel ribs take up the bending moment—is, considering the distance between the steel ribs, open to criticism. On the other hand, the suppositions under which the arches of the Fort Worth viaducts were calculated are definite and logical, and may be summarized as follows: The steel ribs alone carry, in the first stage of the work, their own weight, that

* This discussion (of the paper by S. W. Bowen, M. Am. Soc. C. E., published in August, 1914, *Proceedings*, and presented at the meeting of December 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† St. Louis, Mo.

Mr. Gayler.

of the concrete as it is being poured into the forms around them, and that of the lateral braces. The stresses thus produced in the steelwork can be accurately ascertained; the concrete is merely dead weight, and the question of the difference of the moduli of elasticity of the two materials is not involved. The final stresses in the arch ribs resulting from the additional weight of the superstructure, and from the live loads, are provided for as in other reinforced concrete arches, the steelwork of the ribs forming part of the steel reinforcement, as fully explained by Mr. Bowen.

3. The distribution of the metal through the body of the concrete in the ribs of the Fort Worth arches is far superior to that in any Melan arch ever built.

After careful consideration of the problem with which the consulting engineers had to deal, that is, to build, safely and economically, the arches of the vidaucts under the difficult conditions of a limited clearance, the probability of sudden and extraordinary floods, and an unstable river bed, the writer is convinced that the plan adopted was the one logical solution of the problem. Under analogous conditions, it will undoubtedly be repeated.

The conclusion, however, which Mr. Bowen has reached, that "for high structures, and for those over streams subject to sudden and great variation of water level, this method is cheaper and safer than to use falsework supported from the bed of the stream", needs modification, inasmuch as, in the case of high structures, this method has to compete, not so much with ordinary falsework, as with steel centers placed below the concrete.

Where the height of the arch above the high-water line admits of the use of "free" centers, that is, centers under the concrete, the latter method, in most cases, will be found to be the more economical.

In the first place, the height of the steel centers is then independent of the thickness of the concrete arches or arch ribs, and great saving in the weight of the steel, especially in long spans, can be obtained by choosing an economical height for the steel trusses or arches.

It is also feasible to get along with a set of centers for one (longitudinal) half of the arch only, and to utilize this set again for the other half. On a number of concrete viaducts great saving in the cost of erection has been obtained in thus using the falsework or centering over again, as for instance, on the great viaduct built a few years ago in the Duchy of Luxemburg, on the Walnut Lane Bridge, at Philadelphia, Pa., the Rock River Bridge, at Cleveland, Ohio, and the Kings Highway Viaduct, at St. Louis, Mo. Even the scrap value of the free centers is an item worth taking into account.

The saving in steel gained by utilizing the steel ribs as reinforcement of the arch will thus in most cases be greatly overbalanced if free centers are used. The arches of numerous concrete viaducts

Mr. Gayler.

have been proportioned so that bending moments were practically eliminated and, of course, in all such cases no strengthening of the arch by the embedded steel ribs would be claimed.

Another advantage of free centers is that they are applicable to the circular as well as to the ribbed arch. The method used on the Fort Worth arches, except for short spans, is probably limited to the ribbed arch, on account of its greater available thickness.

An excellent illustration of the advantages of free centers over embedded centers, in the case of a high structure, is found in the successful erection of the Rock River Bridge, at Cleveland, Ohio, by three-hinged steel centers, designed by Wilbur J. Watson, M. Am. Soc. C. E.*

Mr. Bowen states that the object of his paper is "to bring out a discussion of the type of reinforced concrete arch in which the reinforcement is composed of structural shapes and is designed to support the weight of the forms of plastic concrete in the arch ring on ribs", but a few remarks on other features of the viaducts may be allowed.

In looking over the foundations of the piers, as shown on Plates XV, XVI, and XVII, an impression was left on the writer's mind that the old conflict between the sincere endeavor of the engineer to obtain the best results and the barrier set up by a limited appropriation has not been decided fully in favor of the engineer, as far as the question of foundations of the piers of the Fort Worth bridges is concerned. This is written with considerable diffidence, because a thorough acquaintance with the nature of the material in the river bed alone could justify criticism. The following points, therefore, are brought out merely as suggestions, and will be touched on as briefly as possible.

The footings of a number of the piers in the bed of the river, that is, inside of the levee protections, for instance, Pier 3 of the Main Street Viaduct, the piers of the Seventh Street Viaduct with the exception of Nos. 1 and 2, and some of the piers of the East Fourth Street Viaduct, are not carried down as deep as the river bed. For their protection against scouring, rip-rapping is relied on, which may, or may not, be properly renewed in years to come.

The base of Pier 2 of the West Seventh Street Viaduct, judging by the small-scale elevation on Plate XV, seems so provokingly close to rock that it may not be unreasonable to ask why this chance to obtain an unyielding foundation on both sides of a rather flat concrete arch was not utilized.

Even the problem of a solid foundation for Pier 2 of the Main Street Viaduct, carefully as it seems to have been studied by the consulting engineers, leaves some doubt in the mind whether rock might not have been reached at a reasonable price. According to Plates XV

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 1.

Mr. Gayler.

and XVII, the 15-ft. oak piles at the north side of the pier reach down to within a few feet of it.

As to the design of the bearings of the track rails, adopted for reasons of economy, it is not likely to be repeated on other viaducts. The life of the fiber pad, ½ in. thick, which is to act as a shock absorber, will not be a long one, and the hammering of the wheels under the electric railway traffic will then be a serious matter for the slabs and girders.

In speaking of the advantages of the three-hinged, ribbed arch, it was an oversight on Mr. Bowen's part to claim that water-proofing is, in every case, a necessity for the circular arch; but, aside from this, every word he has said is true, and should be welcomed by every structural engineer. Concrete arches should either be built with hinges or efficiently reinforced against temperature stresses. Too long, for the credit of the Profession, have concrete arches been built in the United States with disregard for temperature stresses or under assumptions of changes of temperature which, on their face, are not true.

Changes of temperature in the body of concrete arches produced by long-continued heat or cold can be ascertained, either by judgment confirmed by experience, or by actual tests on existing structures. Whenever excessive stresses result from such changes, hinges should be used.

As the writer expressed it in a previous publication: This (that is, the assumption of inadequate changes of temperature in order to obtain convenient stresses in the arch) is the only instance on record in the history of engineering where an attempt is made to adjust the laws of Nature to the works of man, instead of *vice versa*.

It is true that in former years the three-hinged steel arch had gradually to give way to the two-hinged or to the fixed arch, on account of its lack of rigidity under moving loads, but with the heavy concrete arch this defect is, to a great extent, overcome, and should not be a serious objection to the use of the center hinge.

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PAPERS AND DISCUSSIONS

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THE CLARIFICATION OF SEWAGE BY FINE SCREENS.

Discussion.*

By Messrs. Rudolph Hering, J. H. Granbery, Langdon Pearse, L. C. Whittemore, Samuel A. Greeley, and E. Kuichling.

Mr. Hering.

Rudolph Hering,† M. Am. Soc. C. E.—The author has given the most complete description of apparatus for screening sewage that is published to date. Heretofore, the best description was by Dr. William P. Dunbar,‡ but, being a physician and a sanitarian, he does not enter fully into the engineering features. Therefore Mr. Allen's account of these features of practically all the screens that have been used for the treatment of sewage is welcome.

The speaker has only one criticism to make, and that relates to the title of the paper: "Clarification of Sewage by Fine Screens". Screens do not clarify sewage at all. Even the finest screens do not remove turbidity, and a turbid sewage is not one that has been clarified.

Screens are used for the purpose of, and are of most value for, retaining floating matter. Floating matter may clog pipes or pumps, and it usually makes the surface of watercourses, into which it may be discharged, offensive.

There are in use a number of different designs, which have originated mostly in Germany. Almost every conceivable arrangement has been tried there. Most of the screens are apparently satisfactory; each engineer usually prefers his own. All have some advantages over others, and an outsider, who takes a general point of view, can usually determine the specific conditions which are best suited for one or the other design. The author has done this, and has given his general conclusions.

* Continued from November, 1914, *Proceedings*.

† New York City.

‡ "Principles of Sewage Treatment;" First Edition, Translated by H. T. Calvert, Lond., 1908.

Mr. Hering.

The general conclusions which the speaker has drawn from his study of screens, are substantially as follows: Screens which are to be used before the sewage reaches pumps may be quite coarse; they may be bar screens. It would be throwing away money to use a fine screen, like that of the Riensch-Wurl type, before a pump and for no other purpose, because a pump, and particularly a centrifugal pump, will pass very large solid matter without objection.

Coarse screens are also sufficient for any sewage treatment, when the sewage first goes into tanks where it is supposed to be detained. Fine screens are unnecessary in this case because coarse matter may remain in the tank. The heavy matter will settle to the bottom and the light matter will stay on top. Light matter can be removed much more thoroughly and cheaply from a tank than from a screen.

Fine screens have a different purpose than coarse screens and, therefore, a different line of application. A fine screen is needed when the raw sewage is to go out into a river. A coarse screen would not be satisfactory, because it does not prevent floating matter, visible on the surface of the water, from passing out. Take the case of New York Harbor, or along any river, large or small. If the sewage is to receive no other treatment than screening, a fine screen is necessary.

Fine screens are also desirable if the sewage is to be put through sprinkling filters, or if it is to be turned upon sand filters. By passing sewage first through the fine screens, a much greater quantity can be put on each acre, for oxidation, than is otherwise possible before clogging will result.

Parenthetically, it may be added that in some of these sewage filters even the finest screens may not prevent clogging. The speaker has pulled out from under sprinkler nozzles spreading sewage over a bed, lumps of matter, mycelium and other fungous growths, almost as large as one's fist; and yet the sewage had been fine-screened. Of course, the fungus had grown to large size where it was found, and plants will grow large, even in the sewage beds themselves. Screens do not prevent clogging from such causes, nor will they stop grease from accumulating on and obstructing the beds.

The speaker thinks that in the future there may be another application for fine screens not found in use at present. It refers to the removal of suspended matter before the sewage reaches an outfall. Quite a number of years ago the speaker was shown a result in England which then astonished him. Dr. Travis, in Hampton, having taken some sewage out of a sewer, passed it through filter paper, which made it perfectly clear. He said that this sewage was, not only clear, but on standing it would not get foul. He showed the speaker samples which had been standing for weeks and months, and they had no odor.

Mr. Hering.

The speaker has found the same result in American cities. If you take sewage, as it leaves a house or soon after, and filter out all the suspended matter—in other words, get a clear solution—it very rarely becomes foul on standing.

When sewage leaves the house, nearly all matter which becomes foul is in a solid or colloidal form. There is very little organic matter in solution: a little urine and a little of the animal and vegetable juices in the kitchen waste water will be about all. The great volume of the liquid leaving the house is water. The speaker has had sewage analyzed as it leaves the house and has found that the organic matter, the only matter with which we are concerned here, is nearly all in a suspended condition.

Now, as the sewage travels down the sewer this solid matter is broken up, comminuted, and dissolved, and the liquids leave the animal and vegetable cells, so that, at quite a distance down the sewer, one will find a large quantity of organic matter in solution. The figures that Mr. Allen has given in the tables were obtained by various authorities, and usually give the results at the mouths of the sewers. The speaker has thought that the authorities ought to add how far the sample was taken from the average point of sewage origin, because the farther the sewage flows the more organic matter it will hold in solution.

In the future, as our population increases, there is going to be more trouble or expense than now in treating sewage to obtain higher standards. It may then become more expensive than now to oxidize the stronger solutions of organic matter below the sewage outfalls. It may then be necessary to remove the suspended matter higher up and as near to its origin as practicable, because much of the organic matter which soon dissolves cannot be treated as economically in liquid as in its previous solid form.

Take, for instance, a case like the Passaic Valley Sewer, near New York City. The main sewer is more than 20 miles long. When the sewage of Paterson, at the upper end, gets to the lower end of the trunk sewer, at the pumping station, fine screening is superfluous. There is almost nothing left to screen out except the very coarse and slowly soluble matter which has come down that great distance and must be taken out simply to protect the pumps. Most of the organic matter is churned up and comminuted. It is the same as at the outfalls of the longest sewers in London, Berlin, or Paris. Most of the organic matter which could have been taken out before is still there, but is so finely comminuted that screens cannot remove it.

The speaker believes that where sewage is disposed of by dilution, a removal of the suspended matter nearer the origin may often be economical, and may increase the diluting capacity of the stream.

Mr. Hering.

Besides screening, there is one other way in which one can quickly get most of the suspended matter out of sewage, and that is by settling basins. It is known that more suspended matter can be taken out of sewage by sedimentation than even by the usual method of fine-screening. This may not always be true, however, if the screening is done early, when more screenable matter is floating, than later. Usually, the preference between screening and settling, and the best locations for treatment, will be determined by the question of relative cost for the advantages gained.

The speaker believes, as the author has argued, that the question of screening and its relative advantages should be studied a little more thoroughly than has been done heretofore.

Mr. Granbery.

J. H. GRANBERY,* M. AM. SOC. C. E. (by letter).—The screening methods in the Paris sewers may be of interest. The process has been in use there for some time—since 1867, according to Mr. M. Ducat, Assistant Engineer, *Bureau des Eaux et des Egouts*. With the exception of changes of motive power, and improvements in mechanical details, the apparatus is practically the same as then installed, and has been giving satisfactory results.

The sewers of Paris are of such large cross-section that the screening is carried on in the sewer itself, and as a consequence the material is fresh, and the odor is no more objectionable than that of any underground passage. It is not unlike the odor of the New York subway on a wet day.

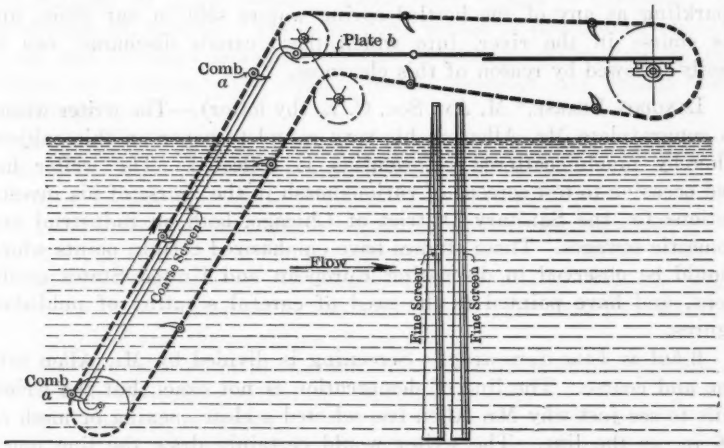
The screening is done in certain portions of the sewers, which have rectangular channels provided for this purpose, and consists of both coarse and fine screening. The device is shown diagrammatically by Fig. 30, and consists of a coarse screen composed of inclined bars about ½ in. apart, followed by one or two finer screens which are removable. A series of comb-like bars (*a*, Fig. 30), having toothed parts which project between the bars of the coarse screen, are drawn upward mechanically, collect the coarse screenings, and drop them on a delivery plate, *b*.

Portions which adhere to the combs may be swept off on to the plate by the operator before the combs again enter the channel. Nearly all the coarse suspended material is at or near the top of the liquid mass. The finer material, which is more evenly distributed, and the material which passes the coarse screen, are caught by one of the fine screens. These are used singly, but the second one is lowered preparatory to removing the one in position, when the latter is to be cleaned. The fine screens are withdrawn and cleaned by hand above the channel.

The apparatus is mounted on a car which is driven by electricity, and the whole is moved against the current while the screening is

* New York City.

being done. This car can pull a trailer which carries a hydraulic press. The latter is used for baling the screenings, which are then taken to the "quai" (or to the railroad yards), whence they are shipped to the agricultural districts and used for fertilizer, or to low land that requires filling in. The Quai de la Megisserie, on the Seine, is one of these loading stations, and the sewer which parallels it has a wide channel in which boats are operated. These boats may be used to transfer the baled screenings along the "quais". Mr. Granbery.



COARSE AND FINE SCREENS USED IN PARIS SEWERS

FIG. 30.

The channels are normally filled to within a foot of the floor, on which carts may be used in certain portions, and this floor opens on the "quai" (in the case referred to) through openings protected by steel doors. These doors have been provided since the "inundation", and are arranged to prevent the water of the Seine from backing up into the sewers, which had caused considerable trouble at that time.

These works are in the heart of Paris, and do not cause any nuisance. There is no surface indication of their presence, and one may pass the loading stations day after day for weeks without becoming aware that the traffic carried is sewage.

The liquid, after screening, is not discharged into the river, as might be supposed, but is pumped under it and discharged by gravity through closed conduits to the agricultural park at Acheres, from 25 to 40 miles distant.

In the park the underground conduits have branches which discharge, through valves operated from the surface by hand, into open

Mr. Granbery.

main ditches, and thence into laterals which are spaced and located to suit the particular crop grown. The liquid discharged has the characteristic whitish color and appearance of dirty dishwater, and an odor that is offensive at the point of discharge but not noticeable 10 ft. from it. The flow in these open ditches is controlled by sluices.

The soil acts as a filter, and the plant growth seems to absorb all deleterious matter, for the soil does not seem to become foul. The filtered liquid is collected by a series of underground drains and open canals and is claimed to be potable. It is certainly as clear and sparkling as any of the bottled spring waters sold in our cities, and its course in the river, into which these canals discharge, can be easily followed by reason of this clearness.

Mr. Pearse.

LANGDON PEARSE,* M. AM. SOC. C. E. (by letter).—The writer wishes to congratulate Mr. Allen on his very complete paper on this subject, whereby much scattered information is collected. The writer has had occasion to test screens of various mesh in the course of his investigations for the Sanitary District of Chicago, both on industrial and domestic sewages. These studies have emphasized certain points which should be observed in discussing European and United States conditions, and have pointed to the need of careful scrutiny of published figures.

What is Fine-Screening?—Screening is divided by Mr. Allen into fine and coarse. The line of demarcation is not exact, but the writer fails to see just why Mr. Allen has selected a clear opening or mesh of 0.6 in. as the line. The writer would certainly draw the line much lower, certainly at or below 0.1 in., in the light of his experiments, as well as those of Monti (quoted in the paper). Screens on small pumping stations designed for automatic service are made with about $\frac{5}{8}$ in. clear opening, and pass most of the fecal matter in American sewages, retaining only rags or coarse non-friable material.

Methods of Analysis.—The writer has long felt that the present methods of studying sewages, from a mechanical standpoint, are not complete. Chemical analyses show nothing of the size of the particles contained therein, and but little of the true strength of a sewage. Biochemical analyses, studying the dissolved oxygen consumption, give truer indications of the real or absolute strength. The mechanical indications would best be given by screening, using screens of different mesh, perhaps in a nest, like sand sieves. The data presented by Mr. Allen on the results of screening show a concordance of results with different mesh which perhaps can be explained if the character of each sewage is defined. Comparative removal by meshes of different sizes must be studied on one and the same sewage, to be conclusive.

* Chicago, Ill.

Mr. Pearse.

Even then it is not always easy to obtain a uniform sewage. Again, samples of both raw and screened sewage must be collected frequently, and averaged for definite periods. Correct sampling, particularly of the crude sewage, is often difficult. The writer has concluded that the best method is to collect samples of both influent and effluent and weigh all the screenings, determining the percentage of moisture. Then the dry material caught is calculated in parts per million and added to the content of suspended matter in the screened sewage to ascertain the quantity of crude sewage. From these figures is calculated the percentage of removal. The suspended matter is determined on both crude and screened sewage—and the percentage of removal is calculated as a check. The former results, however, are usually more consistent.

Screen Factors.—The removal of suspended matter by screening is a complex function having many variables. It depends on:

1.—The size and shape of the mesh or screen;
2.—The loss of head under which the screens are operated;
3.—The character of the suspended matter;
4.—The freshness of the sewage, which in one of domestic origin will be influenced by the area sewered, the time of concentration, and the condition of the sewers;
5.—The dilution of the sewage; *i. e.*, the gallons of flow per capita, or the concentration of suspended matter;
6.—The type of screen.

In the writer's opinion, Table 12 would be increased in value many fold if columns were added giving (1) the area tributary to the screens; (2) the concentration of population per acre; (3) the concentration time of the sewage; (4) the flow, in gallons per capita; (5) the minimum and maximum opening, in inches; (6) the percentage of gross area which is opening; and (7) the content of suspended matter, in parts per million.

Foreign vs. American Sewage.—The great difference in American and European conditions might be shown by the foregoing data, particularly by the suspended matter content. For instance, by referring to Table 10, it will be seen that the Dresden sewage contains 371 parts per million of suspended matter. Many American sewages are more dilute. Industrial wastes are often more concentrated. As a rule, tannery, stockyards, and Packingtown wastes around Chicago seldom exceed 7 500 parts per million, and are usually greater than 500 parts per million. Domestic sewages in the United States seldom exceed 250 parts per million. The average suspended matter at the Thirty-ninth Street Pumping Station during 3 years has been 144 parts per million, from daily samples of twenty-four portions.

Mr. Pearse.

Dr. William Philipps Dunbar* gives the following characteristics of German sewages:

Place.	Suspended matter, in parts per million.	Free ammonia as NH_3.
Hamburg	299.4	27.5
Frankfurt	1 390	31.5
Wiesbaden	74	37

The writer does not believe that Mr. Allen has cited sufficient American data to maintain that the efficiency of fine-screening is equivalent to settling, particularly if fine-screening be defined as 0.6 in. or less for the clear net opening. Of the results given in Table 12, all but three are foreign data, on conditions possibly widely different from those obtaining in the United States. Of the three results in the United States, the Jennings screen is on a peculiar sewage, direct from the stockyard pens, containing much manure, hay, straw, etc. The average content of suspended matter is 340 parts per million. On tests made by the Sanitary District, a removal of 33% was obtained (based on "computed" influent). A 12-hour period of settling removed 64 per cent. Of the other two places quoted, Reading and Brockton, the results at Reading are placed much lower by George W. Fuller, M. Am. Soc. C. E.,† where 15 to 20% removal is noted, instead of 42% as in Table 12. This discrepancy is discussed in the following paragraph more at length.

At Brockton a very concentrated sewage is handled from a separate system. The data in Table 7 are quite inconsistent with the text which follows. From Table 7 the removal of suspended solids is 983 — 282 = 701 parts per million. From the text, in the same interval, 1.4 tons of screenings per million gallons were removed, or 336 parts per million, if the 1.4 tons is dry weight. This would give an efficiency of 54% removal instead of 71.3 per cent. If the weight is wet, the percentage would be much lower. Probably both figures are too high. It would be of interest if Mr. Allen would state the source of the analyses, and explain whether they are more than an average of twelve samples, collected during a few minutes once a month. Judging from reports of the Massachusetts State Board of Health, a removal of 983 parts per million of suspended matter does not represent average conditions, but is extremely high. In 1911, the yearly average was reported as 363 parts per million.

Reading Screen.—The data quoted from Emil Kuichling, M. Am. Soc. C. E., and from "Sewage Sludge" by Elsner, Spillner, and Allen, are not consistent. Mr. Kuichling estimates the dried matter at 750

* "Principles of Sewage Treatment," First Edition, Translated by H. T. Calvert, p. 34.

† "Sewage Disposal," p. 381.

lb. per million gallons, or 90 parts per million, giving an efficiency of 42% on a sewage containing a total of 215 parts per million of suspended matter. Mr. Allen's figures agree with a statement on page 77 of the Annual Report of the City Engineer of Reading (1910), where 31 cu. ft. per million gallons of screenings were obtained with 90% moisture. Taking the weight at 63 lb. per cu. ft.,* 1 953 lb. of wet screenings would be recovered, or 195 lb. of dry screenings per million gallons. This would be equivalent to 23.4 parts per million. Even on a weight of 70 lb. per cu. ft. the equivalent removal is only 26 parts per million. From the City Engineer's report for 1909, the suspended matter in the screened sewage appears to average 151 parts per million. Hence the screening efficiency is $\frac{26}{177}$, or about 14.7 per cent. Mr. Pearse.

The Reading screen was out of service the equivalent of 63 days in 1910, and 77 days in 1911. This would point to the need of duplicate installations.

Scrutiny of Published Data.—It is evident, from a comparison of the data quoted regarding the three American plants, that the efficiencies of screens are probably lower than Mr. Allen's figures. More detailed figures are needed on long-continued tests in order to establish the true figures. If a settling basin is to be rated by its average accomplishment, so should a screen. The writer, therefore, thinks that Mr. Jennings' figure of 63%, based on a peak load, is not a fair one to compare with an average load on a settling basin.

Other Types.—Although not strictly for sewage, two classes of screens may be added, to make Mr. Allen's references complete. One is the rotary or drum screen devised by the United States Reclamation Service.† This was for the purpose of removing trash from an irrigation channel on the Salt River project, and was provided with ½-in. mesh. The other is the type of machinery known in the paper trade as save-alls. These were devised to retain particles of pulp, paper, etc., which would otherwise be carried into a stream or sewer with the wash-water. They are largely of the drum type, fed inside, like the Weand screen, but are sometimes built so that the flow is from the outside inward. Cylindrical screens of this type have been in operation for nearly 20 years. Meshes as fine as 100 per lin. in. are used. They are cleaned by water or by air.

Comparative Tests.—Comparative screening tests on the Thirty-ninth Street sewage are shown by Figs. 31 and 32. This is a domestic sewage, very dilute, containing about 140 parts per million of suspended matter, from a drainage area of 22 sq. miles, with a concentration time of about 5 hours. The dry flow is roughly 200 gal.

* "Sewage Sludge," p. 208.

† Described in *Transactions*, Am. Soc. C. E., Vol. LX, pp. 337-338.

Mr. Pearse.

per capita. The population tributary in 1910 was 274 000, or 19.1 per acre. The tests were all made to a constant loss of head of 0.5 ft. on wire mesh screens, 4.2 sq. ft. in area, having the mechanical characteristics stated in Table 23.

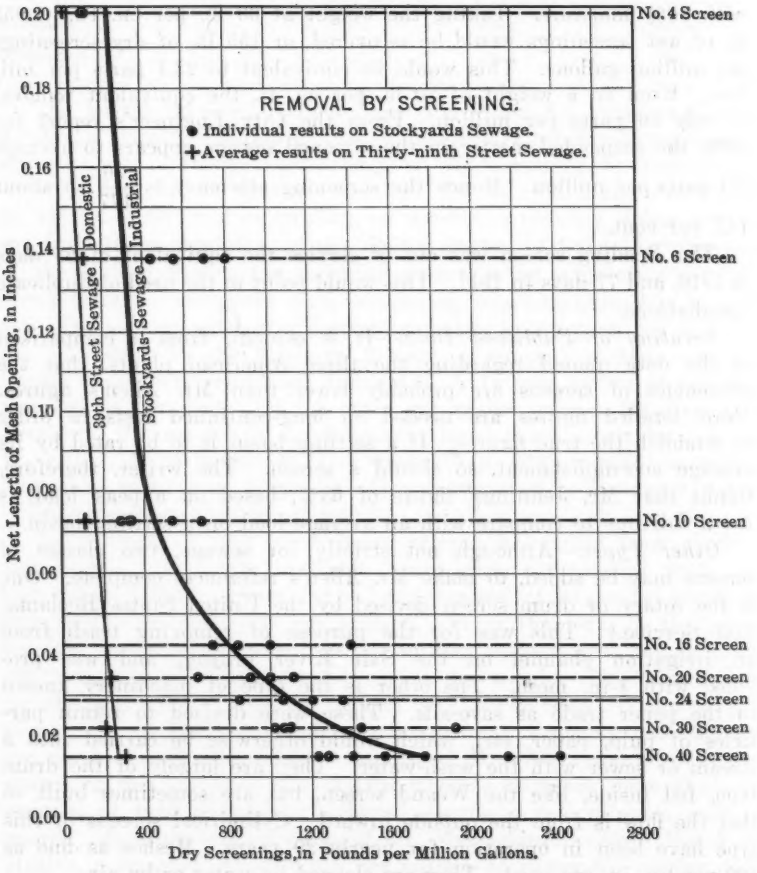


FIG. 31.

Screening tests made with screens of the same mesh, on the Center Avenue sewage, give the results noted by Fig. 31. These were made with a loss of head of 4.5 ft. on a 21 by 24-in. screen. The Center Avenue sewage contains much stockyards and Packingtown waste, with quantities of coarse material, such as paunch manure, hay, particles of meat, hair, etc. The drainage area is 660 acres, with a con-

centration time of about 30 min., a day flow of 700 gal. per capita (29.0 cu. ft. per sec.), and a night flow of 400 gal. per capita (16.4 cu. ft. per sec.). The removal is far higher than at Thirty-ninth Street. Mr. Pearse.

TABLE 23.—PROPERTIES OF FINE SCREENS.

Meshes per linear inch, nominal.	Diameter of wire, in inches.	Net Opening. Length of Side.		Open space. Percentage of gross area.
		Inches.	Millimeters.	
4	0.048	0.198	5.04	65
6	0.034	0.137	3.48	64
10	0.026	0.072	1.83	54
16	0.021	0.042	1.07	42
20	0.016	0.034	0.86	46
24	0.015	0.029	0.74	42
30	0.012	0.022	0.56	41
40	0.010	0.015	0.38	28

Effect on Organic Content.—The writer has been much interested in the data presented relative to the effect of screening on the organic content of sewage, as this problem has been studied by the Sanitary District from the standpoint of the biological oxygen consumed. The results quoted from G. M. Wisner, M. Am. Soc. C. E., were determined several years ago by the methylene blue method on a sewage now found to have a biological oxygen consumption of 100 to 150 parts per million. Recent extended tests on the improvement made by a 30-mesh Weand type of screen on the Center Avenue sewage, with a biological oxygen consumption of 1 100 parts per million, show an improvement of only 6%, with a removal of 8% of suspended matter, from a sewage containing about 487 parts per million of suspended matter. Settling the screened sewage removed 283 parts per million of the suspended matter, making a total removal of 66% on the crude sewage, with a reduction in oxygen demand of 42% in a number of tests. This certainly does not indicate that screening is comparable to sedimentation.

Recovery.—The screenings from the rotary screens, on a packing-house and on the Center Avenue sewer, averaged 84.3 and 86.4% of moisture, respectively, as compared with 91.5% of moisture in the sludge from an experimental Emscher tank 17 ft. deep. The organic content was high, but the fat content was not great, largely because the fats are in a finely divided emulsion, and require chilling before congelation and rising to the surface. The writer is not very confident in general of obtaining valuable recoveries from domestic sewage. Doubtless, too, the material would be open to the criticism that the foreign material, rags, etc., in the screenings would create suspicion in the mind of a farmer purchasing it as a fertilizer.

Mr. Pearse.

Cleaning.—The writer has always favored the use of compressed air instead of water for cleaning rotary screens of the drum type. In a screen tested in Packingtown, of a partly submerged drum type, so much water was used that a settling basin had to be built to settle the screenings. With the Jennings screen, the heavy air pres-

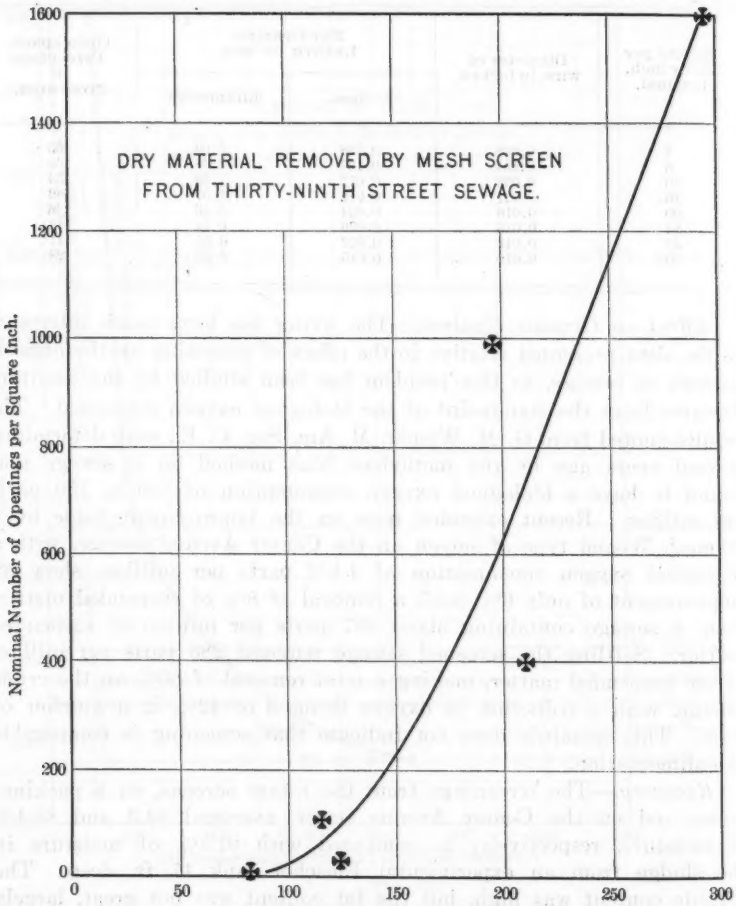


FIG. 32.

sures first used were too severe for the screen fabric, but later reduced pressures have appeared favorable. The cleaning must be adequate to keep down the loss of head or clogging of the screen below defined limits; little is gained in removal after a screen once begins to clog, as the rate of clogging increases rapidly.

Mr. Pearse.

Disintegration.—Mr. Allen refers to the disintegration of solids, with consequent passing through a screen. This would appear to be dependent on their character. Non-friable materials certainly would not break down. Further investigation is required. If pumping is needed, the disintegration produced thereby will probably exceed that caused by the screen. The writer has heard comment by other observers on this point, but would judge that that condition, particularly with the Riensch-Wurl type, is not continuous.

Mechanical Details.—The Weand screen requires considerable loss of head, and consequent pumping cost, in many localities. The Riensch-Wurl and other partly submerged foreign types require but slight loss of head, compared with the Weand.

Of the screens now in use in America, some mechanical improvement is required to put them in a permanent class. The screen at Reading may have been experimental, but it required costly repairs. The screen at Brockton has had some difficulties, as outlined by Mr. Allen. Foreign practice has developed certain types of screens which appear to have been operated with slight mechanical troubles, if any. Doubtless some can be adapted to American conditions. The Jennings screen is very simple, and is capable of being developed to a higher pitch of mechanical construction, after the experimental stage has been passed.

Specifications.—The writer thinks that, for the purchase of screens, the specifications should be drawn to secure a thoroughly satisfactory mechanism with sufficient stability and generous detail to stand the constant wear and tear, particularly if finances preclude duplicate machinery. The removal expected may or may not be a matter of contract. If it is to be guaranteed, it should certainly be defined more rigorously than in a specification recently brought to the writer's notice, in which a removal "equivalent to that of a settling basin" was required.

Results Obtained.—From a study of screening, under working conditions, on industrial wastes, the writer is inclined to believe that from 20 to 35% removal of suspended matter can be obtained with from 30 to 40-mesh screens, under favorable conditions, on certain industrial wastes. He considers, however, the figure of 30% removal, advanced by Mr. Allen, far too high for ordinary domestic sewage, particularly if from a sewer in a combined system with a long concentration time, fed by a large area in which the population uses much water.

Applications.—The writer does not share the enthusiasm expressed by Mr. Allen in his conclusions. Undoubtedly, the use of fine screens for sewage deserves more serious consideration than heretofore, but the results to be obtained are by no means demonstrated as yet. Careful study and skilled tests are required before the art of screening will

Mr. Pearse.

be placed on as accurate and satisfactory a basis as sedimentation is to-day. Comparative results on the same sewage do not indicate that screening, even with a 30-mesh screen, is the equivalent of thorough sedimentation, nor within 50 per cent.

Mr. Allen is right in calling for more fundamental data on efficiencies and costs, and such data will be required before screening will develop widely in the United States. European data are of little value as a general guide to the results to be obtained in the more dilute sewages common in America.

The writer is of the opinion that the application of screening to industrial wastes is a field which has been overlooked. In many industries wastes of a coarse nature are discharged, which do not readily disintegrate. The application of screening in Packingtown, particularly on paunch manure overflows, and in the stockyards, has demonstrated worth. Possibly in tanneries, wool washing plants, and the like, a use may be found under favorable conditions.

In the paper and pulp industry considerable application has already been made to retain pulp which would float away in wash-water. A screen of the drum type has also been tried at Anaheim, Cal., on the waste of a beet sugar works by C. G. Hyde, M. Am. Soc. C. E., with considerable success.

Acknowledgment.—The writer trusts that Mr. Allen will pardon this frank analysis of the situation, as it is thought that it will clear the atmosphere for the future of screening, and encourage the operators of screens to obtain really valuable data. The writer wishes to acknowledge the courtesy of the Trustees of the Sanitary District of Chicago and the Chief Engineer, Mr. George M. Wisner, for material drawn from a forthcoming report.

Mr. Whittemore.

L. C. WHITTEMORE,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read with great interest Mr. Allen's presentation of available data on a branch of sewage treatment practice, as yet little developed in the United States. Extended comment is made by Langdon Pearse, M. Am. Soc. C. E., particularly on the removal efficiencies and reduction in oxygen demand to be expected, as shown by extensive experiments made under his direction for the Sanitary District of Chicago. The limitations of screening in American practice are discussed at length. The writer, therefore, will discuss chiefly two points, wherein fine-screening is likely to be of value in the treatment of packing-house and stockyards wastes, in spite of its obvious limitations as compared with sedimentation. Both have been noticed by Mr. Allen, and are: (1) the possible reduction in rate of sludge accumulation in settling tanks, and (2) the partial or complete prevention

* Chicago, Ill.

of scum formation. The general character of the packing-house sewage is outlined by Mr. Pearse. Mr. Whittemore.

With regard to the first factor, extended investigations have shown that sludge at the rate of 8 to 10 cu. yd. per million gallons may be expected as a result of plain settling in tanks of the Emscher and Dortmund types, based on uniform flow. If corrected to allow for operation at a variable flow, to follow the discharge from the sewer throughout the day, they should probably be increased by 25% or more. With a 30-mesh rotary screen, operating between the hours of 8 A. M. and 4 P. M., an average removal of 950 lb. of dry material per million gallons was obtained. Extending the run to 11 P. M. reduced the average rate to from 320 to about 500 lb. per million gallons. Reduced to sludge of 90% moisture (tank sludges have usually run somewhat higher than this figure), this is equivalent to from 2.0 to 5.5 cu. yd. per million gallons, approximately. Practically nothing was retained between 11 P. M. and 8 A. M., and some reduction in the average removal for the entire day would be necessary. On the other hand, the highest removal occurs during the hours when the flow in the sewer is greatest, and it is evident that material reduction in the rate of sludge accumulation would result with fine-screening. The saving in sludge storage capacity and area required for drying beds, if the sludge is disposed of in this manner, is obvious. A substantial saving in cost of tanks and drying beds should therefore result. Moreover, the screenings are readily handled after slight draining, whereas the tank sludges are liquid, and require several days for drying with the well-digested Emscher product, and considerably longer for other sludges, under favorable conditions. With further removal of moisture, perhaps by pressing, the screenings could probably be disposed of by incineration, as the calorific value of the dry material exceeds 9 000 B. t. u. per lb.

The second point in favor of screening is the prevention of scum formation. Scum has been continuously present in considerable quantities on the various experimental tanks, except where chemical precipitation was used. The high temperature of the sewage (70 to 90° Fahr.) and the rich organic content stimulate vigorous bacterial action, and this, with the high content of grease and light fibrous matter, promotes the rapid accumulation of scum. Comparative experiments in a Dortmund tank in May and June, 1914, under identical conditions (a 3-hour detention period), except that screened sewage was used during the hours of heavy flow in May and raw sewage in June, showed rates of scum accumulation of 0.5 and 3.1 cu. yd. per million gallons, respectively. Scum did not appear at all until the last few days in May, but was continuously present in June, reappearing very shortly after being removed. The difference is striking, and as the scum is neither liquid enough to flow readily, nor sufficiently

Mr. Whittemore.

solid to be easily handled, its removal would entail considerable difficulty in a large installation. The value of scum prevention is very apparent.

The writer fully realizes that his discussion is perhaps rather limited in its application, as the sewage dealt with is of unusual character, but it is hoped that a possible field of usefulness for fine-screening, as an adjunct to, rather than as a substitute for, sedimentation, in treating wastes of this nature, has been demonstrated.

In conclusion, the writer would suggest that percentage efficiencies of removal be scrutinized with care, and the conditions under which they are determined be critically examined before too much credence is given to unusually high removals. A few scattered determinations may lead to an entirely erroneous opinion when based on analysis of small influent and effluent samples by the usual methods. As illustrative of this, it may be of interest to note that, during a 27-day run with the experimental rotary screen, daily efficiencies ranging from an apparent increase of 7% to a reduction of 53% and averaging 32% were found. The screen was operated for 8 hours each day, and influent and effluent samples made up of portions collected every half hour were analyzed by the Gooch crucible method. The rate of flow was constant, and all conditions were particularly favorable for securing accurate results, yet wide differences are noted, and the apparent increase is of course impossible, as a matter of fact. Based on the method of computing the influent analyses, as mentioned by Mr. Pearse, results ranging between 12 and 23% were secured, which are in all probability far nearer the truth. Although the effluent may be affected by errors of sampling and analysis, as well as the influent to a certain extent, it is possible that, when the coarse particles not adequately represented in the analysis are eliminated, the effluent analyses are on the whole more representative, and this method is more trustworthy where the flow and weight of screenings can be ascertained accurately. In relation to this matter, the writer would express the opinion that calculating the efficiencies of screens to tenths or even hundredths of 1% is entirely unwarranted, when consideration is given to the accuracy of the data on which they must necessarily depend. The nearest percentage is undoubtedly well within the limit of error, and is entirely sufficient for practical purposes, without giving a false idea as to the accuracy of results.

Mr. Greeley.

SAMUEL A. GREELEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has been much interested in this comprehensive paper. He inspected the screening plants at Hamburg and Frankfort in June, 1908, and again in September, 1911, at which time he also examined the screening plant at Dresden, then in its first year of operation. It

* Winnetka, Ill.

has been interesting to compare the data secured during these inspections with those given by Mr. Allen. Mr. Greeley.

Hamburg.—The River Elbe divides the city into a larger northern section and a smaller southern section, for each of which there is a screening plant. The older plant described by Mr. Allen is on the river-front, in a very busy, thickly built-up section of the city. Within four blocks are the seaman's school and the entrance to the new tunnel under the river.

The time of flow of sewage to the plant was estimated to be from 2 to 4 hours, depending on the volume. The sewage appeared to be fresh and contained a strikingly large proportion of unbroken fæces.

At the time of the writer's visit in 1911, it was stated that the volume of sewage flowing through the screens was about 50 000 000 gal. per 24 hours. The screens removed from 13 to 65 cu. yd. daily, the average removal being 26 cu. yd. of wet screenings per 24 hours. Mr. Allen states that the removal is 18 cu. yd. per day.

The sludge and screenings are dumped into covered steel barges which are towed to dumping grounds near Neuenfelde, a farming locality along the Elbe, some miles below Hamburg. The tank boats are taken to an unloading station, on a small creek. The station has an inlet for docking the boats, an unloading crane, some small trucks, and a narrow-gauge movable track. The crane lifts the body from the truck to the boat. There it is filled with a shovel, and then replaced on the truck, and run out on the dump. At the time of the writer's visit, the dump was about 5 ft. deep and 75 by 50 ft. in area. It was very foul, giving off a strong pig-pen odor. The material was carted away to the fields in farm wagons.

The country about this unloading station was devoted to truck gardens and orchards, through which the writer walked about 2 miles before coming to the unloading station, and throughout most of this distance the pig-pen odor from the screenings was clearly evident.

Frankfort.—The sewerage of Frankfort is on the combined system. The sewage from the whole city is treated at one disposal plant by screening and sedimentation. As is common in German cities, the sewers are well built and have good slopes, and the sewage comes to the plant in a fairly fresh condition. However, it did not appear to be as fresh as that at Hamburg and Dresden.

The average volume flowing to the treatment plant is about 26 000 000 gal. per 24 hours. The screens remove from 20 to 26 cu. yd. of wet screenings per 24 hours.

The screenings are taken with the sludge from the settling basin to centrifugal drying machines, and the dried material is burned with the city refuse in incinerators. The centrifugal machines are in a large building, and the dried material is handled on conveyors. The

Mr. Greeley.

odors about this part of the plant were very much like those in a garbage reduction building.

It is interesting to note that, up to the latter part of 1909, the sludge and screenings from the Frankfort plant were disposed of on land. This method was unsatisfactory, and the present more costly method of disposal was adopted.

Dresden.—Dresden is sewered on the combined plan. The superintendent at the screening plant informed the writer that the time of flow of sewage from the center of the town to the screening plant was only about 0.5 hour, and for most of the sewage, did not exceed 2.25 hours.

The average quantity of sewage flowing to the plant was 26 500 000 gal. per 24 hours. During storms, this quantity is greatly increased. The superintendent stated that the revolving screens removed from 13 to 16 cu. yd. of wet screenings per 24 hours.

At the time of the writer's visit only one of the disk screens was in operation, and the difference in level of the sewage on the two sides was 11.8 in. The maximum loss of head expected was 23.6 in. The writer was told that the disk screens sometimes required washing with a hose once every hour to remove fat and grease. At other times, one washing on each 8-hour shift was sufficient. One man attended to the ordinary operation of all four screens.

The screenings were taken to long wooden troughs in the yard. These troughs had false bottoms of wood set about 18 in. above the paved floor of the yard. The liquid from the screenings drained to the floor and flowed back in a sewer to the screens. The troughs were roofed over, but not housed in. The dry screenings are said to be good for fertilizing, and are removed by farmers, who pay for the privilege. Although there was only a small accumulation at the plant when seen by the writer, the material was offensive, having the characteristic pig-pen odor.

Percentage of Removal.—It is hard to express the removal of suspended matter by screens as a percentage of the total original suspended matter, because of the difficulty of securing representative samples for analysis. The data on removal, as obtained by the writer, and as given by Mr. Allen, are summarized in Table 24, in which the percentage of removal is shown. This "percentage of removal" is computed from the weight of screenings caught, the percentage of moisture contained in the screenings, and the suspended matter in the crude sewage. The dry material in the screenings, in parts per million, is calculated against the total suspended matter in the crude sewage. Screenings are assumed to weigh 60 lb. per cu. ft.

The percentage of removal at Frankfort, calculated from the content of suspended matter in the crude and screened sewage, is 10 per cent.*

* *Die Frankfurter Kläranlage.*

TABLE 24.—PERCENTAGE OF REMOVAL OF TOTAL SUSPENDED MATTER BY FINE SCREENS. Mr. Greeley.

Plant.	UNDRAINED SCREENINGS, IN CUBIC YARDS PER MILLION GALLONS.		Suspended matter in crude sewage, in parts per million.	Percentage of moisture in screenings.	PERCENTAGE OF REMOVAL:	
	Greeley.	Allen.			Greeley.	Allen.
Hamburg.....	0.52	0.34	300	87	4.4	2.9
Frankfort.....	0.88	2.48	411	85	6.2	17.5
Dresden.....	0.55	0.97	300	84	5.7	10.1

The sewages represented in Table 24 are more concentrated and fresher than is usual in American cities.

Disposal.—Mr. Allen correctly calls attention to the importance of the disposal of screenings when he states that "another and more probable cause for offense lies in the disposal of the screenings". The writer does not consider that Mr. Allen has given sufficient consideration to this difficult problem. The screenings at the plants visited were very unsightly and unpleasant to handle, quickly fouling the cars, conveyors, and chutes, and causing offensive odors. Certainly, when placed on land, the screenings become very foul. Disposal by incineration confined the odors to within the works, but added to the cost of disposal. As the efficiency of screens increases and greater quantities are removed, the disposal problem also increases. When comparing fine-screening with sedimentation, the matter of disposal must be considered.

Summary.—Judging from his investigations, the writer considers that a removal of suspended matter from domestic sewage amounting to 30% is higher than can be expected in ordinary operation and under reasonable limitations of loss of head, cost, present available devices, etc. The available data indicate that a removal of 15% has seldom been reached in practice. The writer's observations lead to the conclusion that the disposal of screenings is attended with the production of odors which may create nuisances, and that adequate methods of disposal of screenings, with reference to cost and efficiency and the location of the place of disposal, are essential parts of an installation of fine screens.

E. KUICHLING,* M. AM. SOC. C. E.—The thanks of all engineers interested in the treatment of sewage are due to the author for his masterly presentation of useful data on the construction and effect of fine screens. It is the first appearance of such comprehensive information in English technical literature, as the subject has received Mr. Kuichling.

* Mr. Kuichling died on November 9th, 1914. This discussion was the last work he did.

Mr. Kuichling.

little attention from English and American engineers until very recently. In Germany, on the other hand, for more than twenty years past, much scientific study has been given to the screening of sewage, with the view of avoiding the great expense of clarification by means of tanks and filters, in places where the effluent could be discharged into relatively large streams without causing appreciable offense; and hence most of the available data found previously were in German publications.

It is claimed that screens with openings from 0.10 to 0.25 in. wide will intercept a large proportion of the floating and suspended solids in sewage, together with the multitude of bacteria adhering thereto and enclosed therein, and that the effluent will consequently be greatly improved in appearance and rendered more easily capable of rapid assimilation by the water of the stream into which it is discharged; also that the removal of such solids will greatly facilitate every other mode of further treatment. Much of this intercepted matter consists of practically non-putrescible substances like paper, cloth, cordage, leather, hair, bone, wood, bark, husks, corks, bread, fat, etc., and the remainder is mostly fresh and dried vegetable matter, such as leaves, stems, parings, rinds, and seeds, together with fragments of meat, fish, membranes, entrails, etc. These latter substances are putrescible, but usually reach the screen in odorless condition. The screenings, therefore, are free from nauseating odors and can be handled inoffensively; they must, however, be removed at frequent intervals and with the least possible comminution, in order to avoid loss of head and defilement of the effluent. To realize these claims, careful design and operation of screens are imperative.

In regard to the quantity of solid matter removed by fine screens, it may be noted that the figures in Table 12 show a range of 0.34 to 5.94 cu. yd. per million gallons, with an average of 1.90 cu. yd., and a range of 0.014 to 0.282 cu. yd. per 1 000 of population daily, with an average of 0.093 cu. yd. Presumably, the reference is to wet material weighing about 1 500 lb. per cu. yd., and containing about 75% of moisture, so that 1 cu. yd. of wet screenings represents about 375 lb. of dry substance. On this basis the aforesaid averages will be 712.5 lb. of dry matter removed per million gallons of sewage, or 85.43 parts by weight per million, and 34.88 lb. per 1 000 persons daily. These quantities are certainly large enough to make a marked improvement in the quality of the sewage, and to reduce materially the volume of troublesome sludge produced by subsequent sedimentation. It can also be inferred that the sludge deposited after the sewage has passed through a fine screen will be much more easily and extensively decomposed by bacterial action than the sludge of unscreened sewage, but comparative experiments in this direction are still lacking.

Mr. Kuichling.

To compare the weight of the dry solids contained in the sedimented sludge of unscreened sewage with that of the screenings, it can be assumed that, on the average, the volume of such sludge is 2 000 gal. per million gallons of sewage; also that the weight of this sludge is 66⅔ lb. per cu. ft., and that it contains 85% of water, so that each cubic foot contains 10 lb. of dry substance. The volume of sludge is thus 267.36 cu. ft. per million gallons of sewage, and hence it contains 2 673.6 lb. of dry matter, while the screenings contain on the average 712.5 lb., or 26.65% of the former. It may be remarked, in this connection, that the plain sedimentation of the sewage of Providence, R. I., during 1912 yielded an average of 1 758 gal. of wet sludge per million gallons, with 9.97% of dry solids; and, if it is assumed that the wet sludge weighs 65 lb. per cu. ft. and contains 6.5 lb. of dry substance, then the said volume of sludge contained 1 527.5 lb. of dry matter per million gallons of sewage. As a fine screen was not used, no further comparison can be made in this case, but, if the aforesaid average of 712.5 lb. of dry matter per million gallons could here be removed by fine screens, the latter would remove 46.65% of that which was removed by plain sedimentation.

The percentage of suspended matter intercepted by a fine screen varies much in different hours of the day, different days of the week, and different months of the year. It is governed to a considerable extent by the nature of the industrial wastes admitted into the sewers, and by the intensity of the rainfall which washes the streets of the city. If there are many shade-trees, and also if the streets are not frequently cleaned, a heavy storm will cause large quantities of leaves, paper, and other débris to enter the sewers in a short period of time, and will also flush out previous deposits in the sewers, thereby greatly increasing the suspended and floating solids. The variation in quantity of matter intercepted by fine screens having meshes or orifices from 0.06 to 0.08 in. in diameter is shown clearly by Metzger's experiments in 1906 and 1910 at Bromberg, Germany.* In the first experiment, with a wire-cloth screen of 0.08 in. mesh, during an entire dry day in 1906, the volume of sewage screened was 1 622 300 U. S. gal., and the weight of the wet material arrested was 8 118.66 lb., thus making an average of 338.28 lb. per hour from an average flow of sewage of 67 596 gal. per hour; and the maximum weight in one hour was 1 334.66 lb., while the minimum weight in one hour was 8.60 lb. In 1910 the second experiment was with a drum screen of thin perforated metal having circular orifices 0.06 in. in diameter; and, during one week, 77 432 lb. of wet screenings were intercepted from 8 402 773 U. S. gal. of sewage, the averages thus being 11 061.7 lb. from 1 200 396 gal. per day; the maximum in one hour was 1 750.45 lb. from 71 064.4 gal. of sewage, and the minimum was so small that the movement

* Described in *Technisches Gemeindeblatt*, Vol. 9 and 13.

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of the screen could be stopped from 1 to 7 A. M. The wet screenings were deposited in a receptacle and weighed, after decanting the supernatant water, and it was found that the thick pulpy matter left contained 34.3% of moisture. On this basis the aforesaid rates of removal will become an average of 6 054.2 lb. of dry solid matter per million gallons per day, and a maximum of 16 183.1 lb. per million gallons in one hour; and by reducing to parts by weight per million, these figures become, respectively, 724.2 and 1 935.8. The cost of operating this screen for a week was at the rate of $2.45 per million gallons, including all expenses for attendance, oil, waste, air compressor, lighting, and electric power, and it is claimed that this rate will be materially reduced in regular operation.

Data like the foregoing, together with those presented by the author, prove conclusively that fine screens are highly efficient and economical in preparing sewage for admission into comparatively large streams where the dilution of the effluent is permissible; and they also indicate that such devices will prove advantageous in cases where a high degree of purification is required, by relieving settling tanks and filters of a large and undesirable burden. It is to be hoped that comparative experiments will soon be made with the same sewage, which will demonstrate the effect of fine-screening on different modes of sewage treatment.

The chief difficulty in screening sewage is in preventing serious obstruction of the orifices, and in cleaning the apparatus in an unobjectionable manner. Many ingenious devices have been invented to keep the submerged portion of stationary gratings clear, and to remove above water the material intercepted by movable screens. In the former class a considerable quantity of suspended matter is unavoidably comminuted and mixed with the effluent, and in both classes the wet mass must be removed from the moving parts after emerging from the sewage. This removal has hitherto been accomplished by combs, scrapers, brushes, and jets of water, steam, and compressed air; but in all such operations some spattering takes place, whereby finely-divided sewage matter gets into the surrounding air which must be breathed by the attendants, and may thus cause sickness. This danger can be obviated by applying a suction apparatus to the wet screenings and removing them in a pipe to a receptacle, without producing spray and polluting the air. Such a device, however, has not yet been developed, probably because an active demand therefor does not exist.

The speaker became interested in cleaning sewage screens by the suction or vacuum method about 9 years ago, and laid the matter before several makers of vacuum cleaners, as well as a number of skillful mechanical engineers. To his astonishment, these men asserted that wet pulpy material could not be removed from a screen

Mr. Kuichling.

in this way, as the suction pipe would soon become clogged. They did not know that many years ago this process was applied successfully to the cleaning of cesspools and privy vaults, and is still in use in unsewered communities; nor that a distinguished Dutch engineer, Capt. Charles T. Liernur, about 40 years ago, applied it to the removal of excreta through an extensive system of cast-iron pipes laid under the streets of Dordrecht and several other small cities, where the matter was delivered by suction to a central station and there converted by evaporation into a commercial fertilizer. These plants were in operation for many years, and are probably still in use; hence the practicability of the method can be regarded as established.

The designing of fine screens for sewage is essentially a problem for mechanical engineers, but as these devices are used only by civil or municipal engineers, a knowledge of the numerous details involved becomes necessary to the latter. It is not an easy matter to compute the stresses in an inclined disk 30 ft. in diameter, like the Riensch screens at Dresden, subjected to the load of wet screenings and possibly a considerable hydrostatic pressure on a part of its surface; yet the clear recognition of these forces is necessary in order to secure durability and correct action. Means must also be provided for varying automatically the speed of the moving parts, so as to conform to the different hourly quantities of suspended matter intercepted, otherwise there will be either a waste of power or an excessive loss of head by obstruction of the orifices.

Reference may also be made to the loss of head involved in the passage of sewage through a fine screen. The author has cited (page 1889*) a coefficient of discharge of 0.4 for the small slots of a Riensch screen, on the assumption that from one-half to one-third of their area is obstructed. It would be interesting to know how this value was reached, as a much larger figure is commonly used in hydraulic computations. Small orifices and narrow slots in a thin plate of metal have a coefficient of discharge of 0.63 to 0.65, but its value may perhaps become less if the orifice is very irregular in shape. Experiments in this direction are lacking, and few observations of the actual loss of head at a partly clogged screen have been recorded. Estimates of 1 ft. loss of head are occasionally found, whereas at Hamburg the actual loss ranges from 8 to 30 in. It is obvious that this loss might be utilized to control the speed of the moving screen, in order to prevent excessive back-water and the development of internal pressure in a closed conduit.

Of the various fine screens described in the paper, the one invented by Mr. Riensch and manufactured by Mr. Wurl seems to have met with general approval. It has the advantage of presenting a large area to the sewage, thus attaining a minimum loss of head; it is

*Proceedings, Am. Soc. C. E., for August, 1914.

Mr. Kutchling.

also easily cleaned, and when built without a submerged bearing, as indicated by Mr. D'Olier, it can readily be lifted from the sewage for convenient examination or repair; the cleaning is done below the level of the floor used by the attendants, and the revolving brushes can be covered with a hood to prevent the escape of spray and particles of solid matter into the atmosphere; there is no appreciable pressing of soft solids through the slots or orifices by the brushes, and the drippings from the intercepted matter before its removal fall only a short distance into the effluent, thereby reducing the evolution of dissolved gases and odors from the liquid by prolonged exposure and agitation.

All these features must be taken into consideration in designing a fine screen for sewage, both in order to secure efficiency and to render the work of attendance as healthful as possible. At its best, such work is unpleasant, and much more will be accomplished if it is made free from the danger of contracting disease. Economy of construction and operation is the final consideration in a successful design.

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PAPERS AND DISCUSSIONS

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WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT.

Discussion.*

By Messrs. C. E. Grunsky and William Schuyler Post.

Mr. Grunsky.

C. E. Grunsky,† M. Am. Soc. C. E. (by letter).—In Mr. Cory's presentation of this subject, which is of vital importance to a flourishing and fast growing western section, there is a marked departure from the ordinary treatment by an engineer of an engineering problem. He has endeavored to present a picture of the local circumstances of growth and development of San Francisco Bay communities and of the factors which influence this growth, in order that due weight may be given to social, industrial, agricultural, topographic, and hydrographic conditions when the question of a water supply for the Greater San Francisco of the future is considered. The result is a paper full of interest to the California engineer, but with only few points that may call forth some comment from the engineer who deals with water supplies from the eastern standpoint.

It is plain that the author favors the development of local sources of water supply to the limit, before going far afield for a mountain supply of a higher grade.

The established company, the Spring Valley Water Company, formerly known as the Spring Valley Water-Works, has a developed supply of 35 000 000 gal. per day. This company is at present supplying in excess of this quantity, but at the cost of a gradual depletion of its storage. In California the rain which produces run-off falls within the consecutive 6 months, November to April. There may be from 2 to 3 years in succession with only 50%, or but little more than 50%, of normal rain.

* Continued from November, 1914, *Proceedings*.
† San Francisco, Cal.

Mr. Grunsky.

In the water-sheds from which San Francisco's present supply is obtained, the normal rainfall ranges from about 15 to 50 in. per annum (the latter covering a few square miles only), and, in the years in which the rainfall is but little in excess of the minimum, which may be as low as 40% of the normal, there is hardly any run-off. Due to these conditions, there is need for storage, either on the surface or underground, of practically a 3 years' supply, if the local sources alone are to be made dependable.

Due to this fact, the supply from near-by sources involves, first, large reservoir capacity, and second, in the absence of filtration, the control of vast areas of water-shed, in order that by such ownership adequate sanitary protection can be afforded. The Water Company, in line with these requirements, has constructed reservoirs on the peninsula south of San Francisco, which have an aggregate capacity of about 28 000 000 000 gal., and is now constructing an additional reservoir on the opposite side of the bay, on Calaveras Creek, a tributary of Alameda Creek, which will more than double this capacity. The Water Company, furthermore, in order to protect the quality of the water, and also to safeguard rights to the underground waters of the Livermore Valley, has acquired and holds the fee to more than 100 000 acres of land, much of which, by reason of use as water-shed or as reservoir area, is made non-available for agricultural and residence purposes. The collecting ground is but slightly removed from the bay margin, and is generally of the low rolling hill type. Nearly if not quite one-half of the present San Francisco water supply comes from wells in the Livermore Valley and from the gravels in Sunol Valley. The Sunol gravels are recipients of the summer flow of Calaveras Creek and of lesser feeders of Alameda Creek, and in a small measure afford sub-surface storage. The water is collected from the gravels in a filter gallery.

The wells in the Livermore Valley near Pleasanton were originally Artesian, but the water-table has been drawn down by pumping, so that all water obtained from this source is now pumped.

The combined supply from the Pleasanton wells and from the Sunol gravels, after being carried across the Bay of San Francisco from the Alameda to the peninsula side, is pumped into the Crystal Springs main, so as to arrive in the city at an elevation of about 165 ft. Certain quantities of this water, according to requirement, are pumped to higher levels.

The water intercepted at Pleasanton and at Sunol is that which under natural conditions would have aided in keeping the Niles gravel cone, the delta cone of Alameda Creek, supplied with water. By this interception the natural period of flow of the creek has been reduced, and in some measure the sub-surface waters of a fan-shaped area expanding with distance westward from Niles toward

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San Francisco Bay have been reduced. This sub-surface supply is drawn upon, as Mr. Cory explains in his paper, by the pumps at Alvarado and elsewhere which supplement the Lake Chabot and other local supplies of Oakland, Alameda, and Berkeley.

It seems probable that the gravels of the Alameda Creek delta cone are being already drawn upon for more water than they can continuously supply. The water-plane near the pumps is lowered below sea level. It remains to be seen how long the draft of the past upon such wells can be maintained without impairing the quality of the water beyond the allowable limit.

The possibility exists, as Mr. Cory points out, of making additions to the developed local supplies, particularly for the cities on the east shore of the bay; but this water development, as in the case of the San Francisco supply, implies the acquisition of large areas of water-shed and of near-by reservoir sites, and, in view of the low, undesirable nature of the collecting ground, ultimate filtration. All this will be at a cost not materially, if at all, less than that of first-class mountain water.

The fact is generally recognized that the population centering around San Francisco Bay cannot depend long on the water from the so-called near-by sources, but must soon go to some remote source for an adequate supply.

The question naturally arises: Which of the remote sources will be the most available? Pumping from the San Joaquin or from the Sacramento River has frequently been advocated, and much is made of the fact that inferior water to that which these streams could provide is made fit for consumption elsewhere by filtration and by treatment with hypochlorite.

Although this may be true, no one who has the choice would advocate the preparation of a filthy water for domestic use when, within the limit of the means at command, a pure mountain water can be made available.

On the basis of an exhaustive analysis, the writer reached the conclusion in 1901 that the cost of developing and delivering a Sierra Nevada supply of water would not be greatly in excess of developing and delivering an equivalent quantity from local sources. The result of his investigations is embodied in a so-called progress report published in the Municipal Records of San Francisco, and his conclusions have been confirmed, particularly in so far as the selection of Tuolumne River as the best source of supply is concerned, by subsequent investigators for the City of San Francisco.

If the city acquires the properties of the Spring Valley Water Company, at the price at which they are to be shortly submitted, it will be at a cost per million gallons of developed water with which the cost of water from the Sierra Nevada, if brought to the bay region

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in a quantity not less than 60 000 000 gal. per day, would compare favorably.

It would be out of place to attempt a review of the studies which led the writer to make the recommendation that the city make application to the Secretary of the Interior for the right to store waste flood-waters in reservoirs to be constructed at Lake Eleanor and at the Hetch Hetchy Valley, the latter on the main branch of the Tuolumne River; but, though making this definite recommendation and thereupon making the application for the city, through the Mayor, the writer has always been a strong advocate of the acquisition by the city of the established water-works, to which the mountain source of supply would be added.

The writer, as City Engineer of San Francisco from 1900 to 1904, was required to report on a water-works entirely independent of the established system, but, in doing so, he called attention to the proper course to be taken by the city. It is needless to say that he has been disappointed in the result of subsequent proceedings.

The city's application for reservoir sites met with opposition which was directed by Spring Valley Water Company officials. These officials succeeded in alarming the two irrigation districts which were dependent on the Tuolumne River as their source of supply, and thereafter the Water Company, the two irrigation districts, and misguided nature lovers, actively opposed the granting of permits to San Francisco to use the two selected storage sites.

Secretary of the Interior Hitchcock advised that Congressional action in the matter should be sought. His successor, Secretary Garfield, recognized San Francisco's rights, but made the permit which he issued subject to certain conditions, the most onerous of which was the one requiring that the lesser reservoir, Lake Eleanor, be developed first. As the permit was revocable, the next Secretary of the Interior, Mr. Ballinger, unfriendly to the San Francisco project, issued an order to San Francisco to show cause why the permit, in so far as it related to the Hetch Hetchy Valley, should not be revoked.

San Francisco was thus put on the defensive and, due to lack of confidence of various of her official departments in each other, placed this case in the hands of an expert called in from the East. The natural result was that thousands of dollars were needlessly expended in the accumulation of a mass of statistical information, and plans were outlined, not for the San Francisco water supply alone, but for a project to divert from the Tuolumne River more than 600 cu. ft. per sec., and to make use of this water until such time as the cities near the bay required it for irrigation in the bay region.

It was pointed out in the resulting reports that the earlier project, for an initial unit of 60 000 000 gal. per day, was far below the yield of Tuolumne River, and that the city should undertake the larger

project in order to perfect its claim to a larger quantity of water. No assent, of San Francisco or of any of the other cities supposed to be interested in the larger project, has yet been obtained, or has ever been asked for. Mr. Grunsky.

This ambitious project when presented to the Secretary of the Interior led to violent opposition on the part of the owners of land naturally irrigable from the Tuolumne River. These land owners were determined that, if any surplus water in excess of the needs of San Francisco were to be used for irrigation, it should not be used on the other side of the Coast Range.

The representatives of the irrigation interests were therefore in Washington at the hearings, and they neatly accomplished the discomfiture of San Francisco. By claiming that the river would not always afford water enough both for irrigation and the municipal requirements of the Bay Cities, 400 000 000 gal. per day according to the tentative project, the city's experts were put to it to show that the water supply was ample. They did this, at least to their own satisfaction, and then the demand was made by the irrigation districts that, this being so, the permit if granted should be conditioned on the irrigation districts first receiving all the water which they could use beneficially before San Francisco takes any, and that, to the extent necessary to protect the districts in such use, they should be allowed to take San Francisco's stored water at cost.

Instead of refusing to surrender first right to all stored water, the city fell into the trap, as can be seen by reference to the "Raker Law" cited by the author, under which certain rights were granted to San Francisco.

The city's representatives at Washington should have insisted on her prior rights. There should have been no concession of rights in the stored water to the irrigationists. The courage to do this was lacking. At the same time there might well have been, as the writer has always advocated, a cordial co-operation with the irrigation districts, because the city, by storing much more water than called for by its immediate needs, would have been in position for many years in the future, probably for 25, or more, to give annually large volumes of stored water free of cost to the districts, thereby saving to them the need of increasing their fall supply by storage of their own.

San Francisco has apparently now waived her prior rights to the stored flood-water of Tuolumne River. When there is a shortage of supply, it will fall on San Francisco, not on the irrigation districts, which, except for the concession made, as explained, would have had no rights whatever in the water stored by San Francisco, provided, of course, that this storage was made subject to the acknowledged prior rights of the districts to the natural flow of the stream to the limit of these prior rights.

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San Francisco, under the Raker Law, is to expend from $500 000 to $1 000 000 on the proposed storage works in excess of her own requirements in order to be in a better position to meet the obligation, which she has apparently assumed, to supply some stored water to the irrigation districts.

San Francisco, under the same law, will be required to pay for the appropriated water, which should be hers without cost, under State law, at the rate of $15 000 per year for 10 years beginning in 1919, thereafter $20 000 per year for 10 years, and thereafter $30 000 per year. The present value of these payments exceeds $300 000.

San Francisco must pay excessively for the Government land occupied by its dam sites, by its reservoirs, and by its tunnels, conduits, power-houses, transmission lines, and the like. For these lands, of little or no value (the city already owns the fee of the major portion of the level land in Hetch Hetchy Valley), there must be given in exchange lands of greater value acquired by the city but not necessary in connection with the project.

San Francisco, at the direction of the Secretary of the Interior, must construct roads and trails in the National Forest Reservation in which Lake Eleanor and the Hetch Hetchy Valley are located, at a cost of from $500 000 to $1 000 000, and must provide for the maintenance thereof.

San Francisco is compelled, too, to develop power and to sell a large part of this power at cost in Modesto and Turlock irrigation districts. Perhaps the cost will be determined so as to include a part of the storage, perhaps not, because storage was essential for the other purposes. The sale of power at cost will be of little if any benefit to the San Francisco water-rate payer. The remaining power will be available for use in San Francisco. Its transmission will be over lines about 150 miles in length. It would probably cost less to develop the same power in San Francisco with oil at 75 cents per bbl. than to develop the water power and deliver it over electric wires. Consequently, although of economic value to society, the power feature is likely to prove a burden, and not an advantage, to the project as a whole.

San Francisco has voted bonds for the construction of a 60 000 000-gal. per day project. The bonds voted amount to $45 000 000. These bonds are not available for the new 400 000 000-gal. per day project based on conditional rights to water and on the requirement that a large part thereof be spent in a commercial venture involving the sale of power. The city is proceeding, however, in the development of the power as though its rights in the matter, particularly in reference to the use of this bond money, could not be questioned.

The Hetch Hetchy bonds were voted in November, 1909. They cover a project, as stated, for a supply of 60 000 000 gal. per day

to be obtained from Lake Eleanor under the Garfield permit. They cover also an expenditure possibly of $7 000 000 or $8 000 000 for a distributing system of reservoirs and pipes in San Francisco. They were voted with the understanding, so expressed in the ballot, that, if the purchase of the Spring Valley Water Company's properties in their entirety were sanctioned at the same time by the voters, only $30 000 000 would be available for the works necessary to make the Tuolumne project the auxiliary supply to the established system. The purchase of the Spring Valley system for $35 000 000 at the same election was lost by a few votes short of the necessary two-thirds. Mr. Grunsky.

At the time the city thus voted bonds, there should have been created a Board of Water Supply, free from politics. This was not done. The administration at that time and the administrations that have followed, would not let go. The Mayor, who names the Board of Public Works, and who practically selects the City Engineer for the Board of Public Works, the Supervisors, with the Public Service and Finance Committees, the City Attorney, the Board of Public Works, and the City Engineer, all have a hand in the game.

There is no programme as yet for any change. Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be for 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone, it is to be inferred that the political management of the city's water project will be continued. Deplorable as this may seem, it is yet possible that competent management will not be too greatly hampered by political obstruction and political direction; and the hope may be expressed that, despite the unfortunate circumstances of the past, San Francisco may yet achieve an adequate and in every way satisfactory water supply. Nevertheless, the danger should be recognized that competent management of to-day under the political system may be followed by incompetent or even corrupt management to-morrow.

It will be seen from the foregoing that the Hetch Hetchy project under the Raker Law is not the writer's Hetch Hetchy project, and that the new project for water for the entire bay region has not yet been given proper standing by the voters of San Francisco, nor has it yet received the necessary study to determine its main structural features.

Meanwhile the city has made endeavor to buy the established waterworks. According to the latest proposition, to be voted on some time next spring, the Water Company will be allowed to retain several thousand acres of valuable land in and adjoining San Francisco, besides some 5 000 acres of land in Livermore Valley. The value of the Pleasanton wells as a part of the system will be reduced because

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the maximum draft on them is to be limited to 15 000 000 gal. per day, and because the ownership of the 5 000 acres will carry with it the right to draw on the Livermore gravels for all the water that may be needed on these lands. As these lands are fertile and lie in a region with a mean annual rainfall of only 15 in., the demand for water by them will in time be such that there may not be much water left in the gravels for San Francisco. It seems unfortunate that in the agreement for purchase the limit to 15 000 000 gal. in a single day has been made, because in the future, when San Francisco has a gravity mountain supply, the pumped water from Pleasanton wells will have chief value as an emergency supply and when used in an emergency there should be no restriction on the quantity to be taken in a single day.

The author discusses the local water supplies as though their development, in advance of a good mountain supply of unquestionably superior quality, was a matter of course. If this is his view, there are others who are not convinced that, even on the score of cheapness alone, the development of the local sources first can be justified, and, when quality, quantity, cost, and reliability of service are all taken into account, it is hard to see how any other conclusion can be reached than that the mountain source is the next one which should be developed. There should be 60 000 000 gal. per day added to the supply at present available, and this should be mountain water.

In summarizing the estimate of the total ultimate safe draft for municipal purposes on near-by sources of supply, the author combines the developed and the undeveloped supplies, referring to the aggregate as obtainable at a reasonable cost.

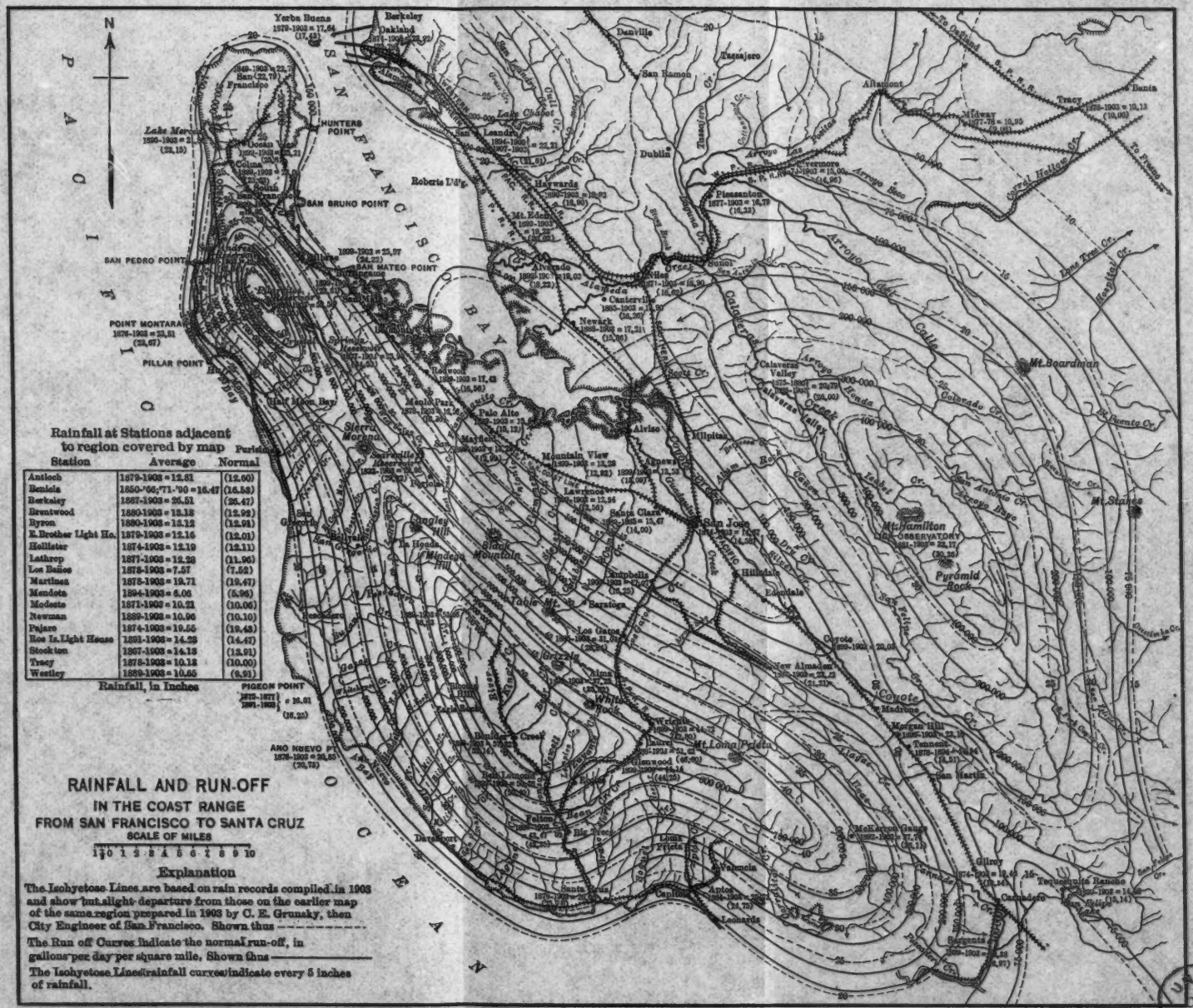
He includes as additional sources all or in part, though perhaps not at the estimates noted in Table 6, which are those of Messrs. Chittenden and Powell.

TABLE 6.

The Peninsula Coast streams, at	50 000 000	gal.	per	day
" Ravenswood and Alviso Wells, at	20 000 000	"	"	"
" Calaveras and San Antonio Branches of Alameda Creek, at	90 000 000	"	"	"
" Area tributary to Livermore Valley, at	40 000 000	"	"	"
Total	200 000 000	"	"	"

Of this quantity (which no doubt includes some over-estimate), only about 15 000 000 gal. per day are actually developed. In the development of the remainder, as already stated, the cost—per million gallons of daily supply—may be as much as, or more than, that of the water from the Tuolumne River, if brought down in a first unit of at least 60 000 000 gal. per day. Some of this remainder will not be obtainable except at the expense of other communities or at a cost

PLATE XLI.
PAPERS, AM. SOC. C. E.
DECEMBER, 1914.
GRUNSKY ON
WATER SUPPLY OF SAN FRANCISCO.


N
PACIFIC OCEAN
SAN FRANCISCO BAY
Yerba Buena 1876-1903 = 17.64 (17.43)
Berkeley
Oakland 1871-1903 = 22.50
San Francisco 1849-1903 = 22.7 Bars (22.79)
Lake Merced 1895-1903 = 2 (22.18)
Ocean 1892-1903 = 23.21 (23.30)
Colma 1889-1903 = 21.
Alameda
HUNTERS POINT
SAN BRUNO POINT
SAN PEDRO POINT
1889-1903 = 25.97 (24.22)
SAN MATEO POINT
POINT MONTARA 1878-1903 = 23.51 (23.67)
PILLAR POINT
Half Moon Bay
Redwood 1854-1903 = 17.43 (16.56)
Menlo Park 1878-1903 = 15.4 (15.32)
Palo Alto 1892-1903 = 15. (15.12)
Mayfield 1867-1903 = 14.
Alviso
Mountain View 1893-1903 = 13.28 (12.82)
Lawrence 1897-1903 = 12.94 (12.56)
Santa Clara 1858-1903 = 13.67 (14.00)
San Jose 1868-1903 = 14.67 (14.56)
Milpitas
Agnews 1893-1903 = 13.52 (13.07)
Danville
San Ramon
Dublin
Pleasanton 1877-1903 = 16.79 (16.33)
Livermore 1872-1903 = 15.00 (14.96)
Altamont
Tracy
Byron 1878-1903 = 10.13 (10.06)
Midway 1877-78 = 10.95 (9.61)
Hayward 1894-1903 = 22.21 (21.61)
Mt. Eden 1892-1903 = 19.22 (19.06)
Alvarado 1892-1903 = 18.63 (18.22)
Niles 1877-1903 = 15.99 (15.69)
Irvington 1877-1903 = 15.99 (16.20)
Mission San Jose 1868-1903 = 15.99 (16.20)
Newark 1868-1903 = 17.21 (16.76)
Centerville 1893-1903 = 15.99 (16.20)
Calaveras Valley 1875-1903 = 24.79 (25.00)
Mt. Boardman
Mt. Stanford
Mt. Hamilton LICK OBSERVATORY 1881-1903 = 28.17 (30.28)
Pyramid Rock
Hillsdale
Edenvale
Campbell 1893-1903 = 23.67 (23.23)
Saratoga
Los Gatos 1894-1903 = 31.72 (32.94)
New Almaden 1903 = 20.42 (21.21)
Coyote 1889-1903 = 29.05
Coyote Madrone
Morgan Hill 1895-1903 = 22.16
Tennant 1879-1896 = 24.14 (24.51)
San Martin
McKeever Gauge 1892-1903 = 17.71 (18.13)
Gilroy 1874-1903 = 19.49 (19.44)
Telegraph Ranch 1892-1903 = 14.69 (14.14)
Pacheco Pass
Mt. Diablo
Walnut Creek
Concord
Clayton
Martinez
Pacheco
Port Costa
Benicia
Vallejo
Napa
Sonoma
Petaluma
Santa Rosa
Healdsburg
Cloverdale
Point Reyes
Bolinas
Tomales
Duncans Mills
Bodega
Valencia
Aptos 1894-1903 = 22.72 (21.72)
Capitola
Santa Cruz 1878-1903 = 30.
Davenport
Bonny Doon
Felton 1892-1903 = 45.47 (45.21)
Big Trees
Boulder Creek
Ben Lomond 1892-1903 = 30.67 (30.69)
Mt. Loma Prieta
Redwood (44.23)
Wrights 1892-1903 = 44.72 (45.46)
Summit 1892-1903 = 51.62 (46.90)
Los Gatos Creek
Guadalupe
Almaden
Lick House
Alameda Creek
Arroyo Mocho
Arroyo Las Positas
Arroyo Valle
Arroyo Seco
Arroyo del Hambre
Arroyo Honda
Arroyo Hondo
Arroyo Aguague
Arroyo Bayo
Arroyo Pajaro
Arroyo Corte Madera
Arroyo Agua Caliente
Arroyo Calaveras
Arroyo Del Puerto
Arroyo Del Valle
Arroyo Del Cierbo
Arroyo Del Rey
Arroyo Del Pescadero
Arroyo Del Rio
Arroyo Del Rio Frio
Arroyo Del Rio Seco
Arroyo Del Rio Chico
Arroyo Del Rio Grande
Arroyo Del Rio Pequeno
Arroyo Del Rio Verde
Arroyo Del Rio Blanco
Arroyo Del Rio Negro
Arroyo Del Rio Amarillo
Arroyo Del Rio Rojo
Arroyo Del Rio Azul
Arroyo Del Rio Morado
Arroyo Del Rio Gris
Arroyo Del Rio Pardo
Arroyo Del Rio Naranja
Arroyo Del Rio Violeta
Arroyo Del Rio Rosado
Arroyo Del Rio Celeste
Arroyo Del Rio Turquesa
Arroyo Del Rio Esmeralda
Arroyo Del Rio Dorado
Arroyo Del Rio Plateado
Arroyo Del Rio Cobrizo
Arroyo Del Rio Bronceado
Arroyo Del Rio Ocre
Arroyo Del Rio Sepia
Arroyo Del Rio Siena
Arroyo Del Rio Umbra
Arroyo Del Rio Tierra
Arroyo Del Rio Arena
Arroyo Del Rio Grava
Arroyo Del Rio Piedra
Arroyo Del Rio Roca
Arroyo Del Rio Montana
Arroyo Del Rio Valle
Arroyo Del Rio Llano
Arroyo Del Rio Meseta
Arroyo Del Rio Altiplano
Arroyo Del Rio Puna
Arroyo Del Rio Paramo
Arroyo Del Rio Tundra
Arroyo Del Rio Glaciar
Arroyo Del Rio Nieve
Arroyo Del Rio Hielo
Arroyo Del Rio Escarcha
Arroyo Del Rio Rocio
Arroyo Del Rio Lluvia
Arroyo Del Rio Granizo
Arroyo Del Rio Tormenta
Arroyo Del Rio Relampago
Arroyo Del Rio Trueno
Arroyo Del Rio Rayo
Arroyo Del Rio Huracan
Arroyo Del Rio Tornado
Arroyo Del Rio Ciclon
Arroyo Del Rio Tifon
Arroyo Del Rio Monzon
Arroyo Del Rio Brisa
Arroyo Del Rio Viento
Arroyo Del Rio Aire
Arroyo Del Rio Atmosfera
Arroyo Del Rio Cielo
Arroyo Del Rio Nube
Arroyo Del Rio Niebla
Arroyo Del Rio Bruma
Arroyo Del Rio Neblina
Arroyo Del Rio Vapor
Arroyo Del Rio Humedad
Arroyo Del Rio Sequedad
Arroyo Del Rio Calor
Arroyo Del Rio Frio
Arroyo Del Rio Templado
Arroyo Del Rio Caliente
Arroyo Del Rio Helado
Arroyo Del Rio Congelado
Arroyo Del Rio Derretido
Arroyo Del Rio Evaporado
Arroyo Del Rio Condensado
Arroyo Del Rio Precipitado
Arroyo Del Rio Lluvioso
Arroyo Del Rio Seco
Arroyo Del Rio Humedo
Arroyo Del Rio Mojado
Arroyo Del Rio Empapado
Arroyo Del Rio Inundado
Arroyo Del Rio Desbordado
Arroyo Del Rio Crecido
Arroyo Del Rio Menguado
Arroyo Del Rio Bajo
Arroyo Del Rio Alto
Arroyo Del Rio Medio
Arroyo Del Rio Profundo
Arroyo Del Rio Somero
Arroyo Del Rio Ancho
Arroyo Del Rio Estrecho
Arroyo Del Rio Largo
Arroyo Del Rio Corto
Arroyo Del Rio Recto
Arroyo Del Rio Curvo
Arroyo Del Rio Sinuoso
Arroyo Del Rio Meandrico
Arroyo Del Rio Trenzado
Arroyo Del Rio Ramificado
Arroyo Del Rio Confluente
Arroyo Del Rio Afluente
Arroyo Del Rio Tributario
Arroyo Del Rio Principal
Arroyo Del Rio Secundario
Arroyo Del Rio Terciario
Arroyo Del Rio Cuaternario
Arroyo Del Rio Quinario
Arroyo Del Rio Senario
Arroyo Del Rio Septenario
Arroyo Del Rio Octonario
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Mr. Grunsky.

to society, which is as necessary to consider as that charged by the author to a Sierra supply.

He includes also from 12 000 000 to 13 000 000 gal. as being furnished by pumping from the Niles gravel cone, and is right in questioning the permanency of a draft of this quantity, and he includes a large development from small creeks in the hills to the east of Oakland, aggregating 21 000 000 gal. per day, and at present developed to but a small extent by the San Leandro Reservoir. When the cost of filtering is considered, and the cost to society, by reason of the occupation of valuable territory, these sources will probably be found to be as expensive as an adequate Sierra supply.

Whether or not this is actually the case, the fact remains that a 60 000 000-gal. first unit from the Tuolumne is within reach, from the standpoint of cost. The development of additional local sources is to be discouraged and not encouraged. The high-grade mountain water should be made available before the possible but improbable development of the nearer sources is awaited.

The writer's conviction remains unshaken that the private concerns on both sides of the Bay of San Francisco should be bought out by the people, and that the entire bay community should then join in perfecting and carrying out as speedily as possible the Tuolumne project, but with suitable modification of the Congressional permit which, unfortunately, is not alone for San Francisco, but also for all Western States, in this, that it establishes a dangerous precedent.

The water supply problems to the northward of San Francisco Bay are separate and distinct from those of San Francisco and the East Bay cities, and need not be brought under discussion in connection with the larger San Francisco problem.

The author presents a diagram, Fig. 15, supposed to represent the latest information relating to rain distribution in central portions of California. This should be used with caution. The precipitation does not increase from north to south in the Sierras, as this diagram indicates, and there is no point known to the writer where, south of the latitude of San Francisco, the precipitation on the western slopes of the Sierra Nevada approaches 80 in. per annum, as shown by the curves of this diagram.

The same suggestion applies to the run-off diagram, Fig. 16. Dependence on the curves there shown, particularly for the Sierra Nevada eastward from San Francisco, might lead to disappointment.

As a contribution to the rainfall and run-off information in the San Francisco Bay region there is submitted herewith the result of a study made by the writer several years ago.

The run-off curves on Plate XLI will show at a glance the limit of the water crop at any point under consideration. Where, as in the

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case of the Crystal Springs reservoir of the Spring Valley Water Company, the storage capacity is so large that practically no water goes to waste, the run-off shown by the curves, less the loss from the reservoir by evaporation and seepage, will determine its productiveness. The curves afford a quick first approximation of the possible limit of yield.

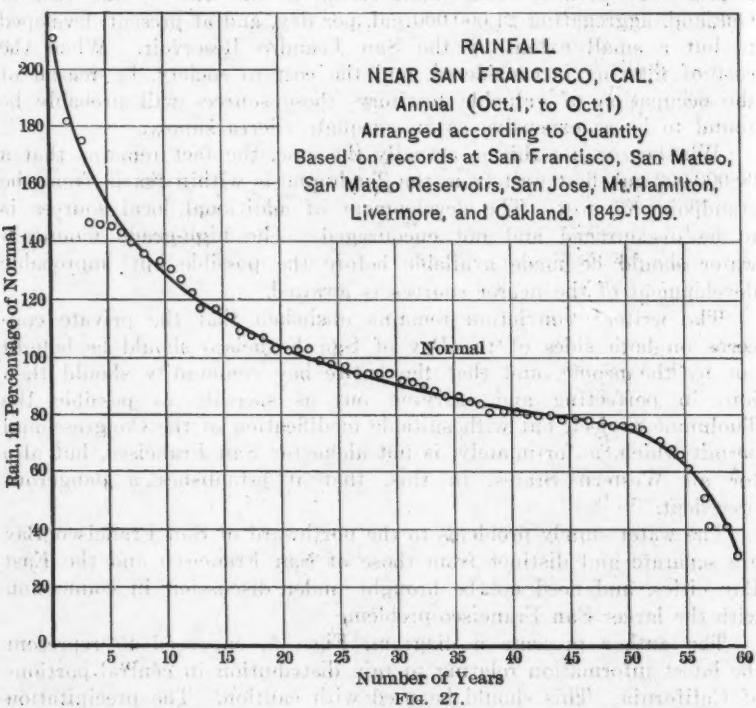

FIG. 27.

The frequency of years of heavy, moderate, and light rainfall in the vicinity of San Francisco are shown by Fig. 27. The rainfall is expressed as a percentage of the normal annual rain in order to give the diagram wider use than just for the locality for which prepared. These data can readily be fitted into a probability diagram to obtain departure from the ordinary law by any one interested in this climatic peculiarity.

Mr. Post.

WILLIAM SCHUYLER POST,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is an inspiration. It indicates to the engineer a legitimate field which he has abandoned and left to the cultivation of promoters and chambers of commerce. All have smiled at the vagaries

* San Diego, Cal.

of the western "booster", and admitted that people were fools to believe; but we have "stood for it", and have not realized that here is a real science of economic importance, one which the engineer is competent to pass on, and which he should pass on—the assigning of values to elements of civic growth, assigning weights to these values, and making a logical deduction from the known to the predicted. Mr. Post.

It is an ordinary platitude that a city must look 50 years ahead in its water supply. The simple matter of width of streets is left to the accident of original dedication. The cost of future congested traffic and the economic wastes of inadequate or over-adequate present construction and planning all belong to a group which invites the intellectual and imaginative qualifications of the engineer, wherein, working in the present, he foresees, provides for, and harmonizes, the economic situation a decade or more beyond.

As an Angeleno, and with a more or less profound geologic knowledge, the writer cannot admit that the San Francisco fault line runs near Los Angeles. In fact, he believes that the nearest active fault is 60 miles distant, and earthquake disturbance is in reality negligible in Los Angeles. However, the writer enjoys admitting that the eastern colonist puts all California in the earthquake class, and balances it in his mind with cyclones in the Middle West. Therefore, as far as the psychology of the colonist is concerned, before he starts for the coast, the conclusion of Reason 12 is sound; and therein lies the interest of such investigations as these.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE VALUATION OF PUBLIC UTILITY PROPERTY.

Discussion.*

By MESSRS. ALEX DOW and F. LAVIS.

ALEX DOW,† M. AM. SOC. C. E. (by letter).—Mr. Gandolfo, after noting the existence of a lack of unanimity among engineers, financiers, and the judiciary, asks: Mr. Dow.

"For example, in making a valuation, should actual cost to date, cost of reproduction new, or fair or market value, be used?"

Why the question? A valuation is a valuation, and a statement of cost is a statement of cost. The two are not the same, and there should be no confusion between them in the mind of an engineer. A statement of what a property has cost can be prepared by a competent accountant from the books of the company, unless the books have been wilfully falsified. A valuation is an engineer's work, and the manner of making it is properly a subject of discussion by this Society. The problem is: Given a certain property, required a valuation. The answer is: An itemized list of the property on an assigned date, and the value thereof, as of the same date, as found by the competent engineer.

It may be that the engineer's instructions are that he shall determine the cost of a certain property, to its present or past owners. In that case, no engineering knowledge of values is required, or of use, except as a check on the accuracy of the cost figures shown by the record; and an engineer accepting such instructions presumably either has himself the necessary knowledge of accounts, or will secure competent assistance for the fulfillment of the instructions accepted. In-

* This discussion (of the paper by J. H. Gandolfo, Assoc. M. Am. Soc. C. E., published in October, 1914, *Proceedings*, and presented at the Meeting of November 18th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Detroit, Mich.

Mr. Dow. structions to make a valuation or appraisal, however, do not require, except collaterally, the ascertainment of cost, and instructions to ascertain cost do not call for valuation or knowledge of values. Why the confusion between the two which seems to exist in the author's mind and in those of many other engineers?

On page 2452,* Mr. Gandolfo says:

"In any valuation for the purpose of rate-making, or for regulating the issuance of securities, actual cost to date is the final figure that should be used."

How is the purpose of a valuation related to the ascertainment of value? Or is it related at all? In such talk, as to the purpose of valuation, are we not departing from the engineering standard which requires an engineer to tell the truth as he finds it, without fear or favor? Are we not setting up a false standard permitting that an engineer's opinion of value shall be different when his client proposes to buy, from what it is when he proposes to sell? Is it not the duty of an engineer called on to make a valuation to say that he finds certain property the present value of which is thus and so? If the engineer's opinion is asked as to what would be a wise price to be accepted or offered for that property; or if he is appointed arbitrator to fix a price; or if he is asked to express an opinion as to a fair rate of return to be allowed on the property in a rate-making case—under any of these conditions, an opinion as to price or as to rate of return is proper; but, to inject his opinion as to price or as to rate of return into his findings of value is to depart from his direct function as an engineer telling what he has found and at what he appraises its present value for its use or purpose; and, to confuse opinion as to price, or as to rate of return, with value of property as it exists, is darkening of counsel.

In the writer's opinion three-fourths of the existing confusion tending to discredit engineering reports of public utility values arises from the failure (or the refusal) to recognize existing value and recorded cost as two distinct things; the other fourth arises from permitting the interest or intention of the client to influence the report of the engineer as to findings of fact. The way out of the confusion is: First, to recognize the fact that value is value and cost is cost, and that the setting of a rate of return or a permissible price is a function outside of, and beyond the making of, a valuation, or the ascertainment of cost; and, second, to recognize and observe fully the distinction between an engineer's appraisal of value as found, and the opinion or argument of a retained expert.

Mr. Lavis. F. LAVIS,† M. AM. SOC. C. E.—The avowed purpose of this paper is to demonstrate that "the proper figure to be used [for the valuation of public utility property] is the actual cost to date", questions of

* *Proceedings*, Am. Soc. C. E., October, 1914.
† New York City.

fraud, extravagance, mismanagement, etc., being supposed to be eliminated. Mr. Lavis.

The speaker does not agree with this, and, considering the very able paper by Mr. Alvord,* it hardly seems necessary to go into any very extended argument to attempt to show that the "actual cost to date" method is impractical, and that the "cost of reproduction" method appears to be the only one which most nearly complies with all the necessities of the situation.

Mr. Alvord points out the necessity of keeping clearly in mind the distinction between cost and value, and it seems to the speaker that as far as any part of the discussion may be applied to the valuation of steam-operated railways, and particularly to those engaged in interstate commerce, it would appear at once that a valuation based on the actual cost to date method, even though it were possible to apply it, would by no means necessarily give a true estimate of the "present-day value."

One must also not lose sight of the fact that, according to most of the Court decisions, and for the purposes of most valuations, the present-day value is further limited and defined by the requirement that this shall be "the value of the property for the purpose for which it is being used", and it is keeping this in mind which makes it sometimes difficult to see why depreciation should be deducted from the reproduction cost of a machine which is working at 100% efficiency.

Of course, we need not lose sight of the fact that much assistance can be derived from knowledge of actual costs in determining present-day values, or the fact that it is the duty of those charged with the determination of present-day values to use such information for all it is worth. Every railroad engineer of experience, however, knows the difficulty, not to say in many cases the absolute impossibility, owing to the lack of records, of determining for many—even for most—of the important railway systems what the actual cost to date is or what the actual cost of even a reasonable percentage of its component parts has been.

Take, for instance, the very simple and common case of a road which had originally been built as a single-track line and afterward widened, without introducing any questions of changes of gradient or location, though these have almost always been present to add an additional complication. Widening a cut of a line already in operation may cost more for the excavation than it would to have taken it out altogether in the first place, by reason of interruptions of traffic and other difficulties, or it might cost less, by reason of the fact that the presence of the existing line afforded better and easier facilities for doing the work. The value of such a cutting to-day, however, is not

* *Proceedings*, Am. Soc. C. E., for October, 1914.

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changed in any way by the fact that it was all taken out 20 years ago, or part of it 20 years ago and part of it 10 years ago, though these facts most probably did affect its cost.

It is quite true that individual cases might be cited to show that in instances similar to that just referred to the actual cost to date method might be a true measure of the present-day cost, but that hardly alters the fact that, in general, it probably is not, or at least not necessarily so. Arguments can be advanced and concrete instances quoted to show the injustice, under certain conditions, of any method of valuation as applied to individual cases, but, admitting that valuations of railways have to be made, and especially looking at the matter from the standpoint of the Interstate Commerce Commission, some general method must be adopted which will be equally applicable and fair to all the railways of the country, to determine with a reasonable degree of accuracy and fairness their present-day value, which may or may not be the actual cost to date, and most likely would not be.

So many arguments and concrete cases bearing on this phase of the subject have already been published in the *Transactions* of the Society, that it seems only necessary to refer briefly to the matter in this discussion in order that Mr. Gandolfo's statements may not become part of its official records without some protest.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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FUNDAMENTAL PRINCIPLES OF PUBLIC UTILITY VALUATION.

Discussion.*

By Messrs. Lewis M. Haupt, L. L. Jewel, Richard T. Dana, F. Lavis, Louis L. Tribus, Joseph Mayer, and Philip Burgess.

Mr. Haupt.

Lewis M. Haupt,† M. Am. Soc. C. E. (by letter).—This paper is an able and timely exposition of a vital question affecting the public weal and concerning which the Engineering Profession has much to do.

The author has stated the fundamental elements clearly, as being legal, economic, and constructive, and has given the definitions necessary to reach clear and just conceptions of their relations to one another as well as the constitutional basis of an equitable settlement for private property taken for public use, under the right of eminent domain.

Yet it seems there are other factors not so readily ascertained, which enter into the solution of the just value. These factors are psychological and are influenced largely by the relations of those who take part in such adjudication. These the author has classified under six heads, the first three of which he rejects as useless and the next two as of doubtful expediency, depending on the trend of their bias. The sixth is the only condition under which one would be apt to render an impartial judgment, that is, the "unbiased and analytical mind that has had ample experience in representing both sides of the question."

In other words, the judge must be competent to determine present values of property as well as franchises, and be a follower of the Golden Rule. Thus, there is introduced the fourth fundamental ele-

* This discussion (of the paper by John W. Alvord, M. Am. Soc. C. E., published in October, 1914, *Proceedings*, and presented at the meeting of November 18th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Philadelphia, Pa.

Mr. Haupt. ment of morality as over against policy, which is so often a dominant factor.

In practice how often does it happen that the evidence is tainted with the desire to win the case at all hazards, or that the witnesses—employees—are suborned by the fear of losing their positions if the truth be told, on the theory that "whose bread I eat, his song I sing"; or the impression that to depreciate value and force the holders of private property to sell at a bargain-counter price is a popular measure and will be sanctioned by the public at large, or that the Government consists of those executives who may be temporarily in power and who are paid to drive hard-and-fast bargains with the people, even to the destruction and confiscation of personal or corporate property, at public expense, which the Fifth Amendment was expressly designed to prevent.

One has only to examine the records of the acquisition of the private waterways and other utilities of the country, or on the Isthmus, to note that the first effect of such a measure is to depress the market values of the securities of prosperous "going concerns", and that in some instances official appraisements are made on depreciated unit prices or reduced quantities of materials, and claims are made for services, or threats that free, competitive, parallel routes will be constructed out of the public treasury, at much greater cost, unless the parties who have invested their funds in good faith and are dependent on returns of interest for their subsistence, consent to surrender for less than half the appraised value, as has recently been attempted.

Surely such methods cannot receive the sanction of honorable engineers much less of a righteous nation, yet they have been proposed but have failed in some instances.

In a democracy, such as this, it would seem that what is just for the whole people as sovereign should be equally just for each individual unit, and that if a statute runs against the individual or the corporation, it should also hold against all the people, who are "the Government." The monarchical fiction holds that "the King can do no wrong" and, therefore, excepts the sovereign from the penalties which it compels the individual (who is far less able) to pay, and he has no redress at Court, and relief by legislation is a vain hope.

A Definite Method.—As to the difficulties of ascertaining the present worth of a public utility, it may be apropos to call attention to the acquisition of the property of the Monongahela Navigation Company which, for many years, was a private enterprise furnishing a means for providing the great manufacturing districts in and around Pittsburgh with coal, at very low figures, until it was thought best to emancipate the tolls by having the Government acquire the property and operate it at the expense of all the people. The argument

Mr. Haupt.

presented was that the Government had developed an open waterway on the Kanawha in competition with the Pittsburgh District which was subject to tolls and paid a tax to the State for its franchise.

The Company was inaugurated under a Pennsylvania State charter on June 14th, 1836, and developed "an immense commerce." The Act provided that the works might be purchased after 25 years by paying the expense of construction and repairs, with the net dividends, with 8% interest thereon. The work was actually completed in 1884. The Government Board found (in 1895) that "the money value of the commerce is very great and far in excess of that of a majority of streams upon which the United States, under its policy of fostering commerce, has expended millions of dollars in improvements."

The President of the Company states that "previous to the recent agitation of the question of purchase by the United States the stock sold at a premium of 80 per cent. * * *, making the present value of the Company's improvement, on a basis of 4½% interest, over $4 000 000." Concerning which the Board states that the number of shares and their market value being variable, it was unable to fix on any amount which properly represents such value at any future time.

The books of the Company show the cost of construction and repairs to have been $3 033 531.34. The Board found "the present intrinsic value of existing works about $1 950 000", exclusive of the franchise.

On October 12th, 1886, President Moorhead stated to the Board, "we wish to be left in the undisturbed possession of our property, and protest that no public interest can be subserved by its appropriation by the United States; that the cost of coal to the consumer will hardly be appreciably diminished, while the annual charges to the United States would be burdensome and increasing, as permanent repairs would be required * * *. It would constitute class legislation of the most objectionable kind. The rates of tolls are the lowest charged on any canal or slack-water improvement in the United States and so low that to take them off altogether would not lessen the price of coal 1 cent per ton * * *. The Government cannot take this property without making compensation, which is the full value of the property of the Company."

No dividends were declared between 1839 and 1853, during which time the construction was in progress.*

These negotiations were continued from 1884 to 1907, when the final award was made, by a jury, for the sum of $3 761 615.46, after 23 years of delay. At this date the value of the works and franchises had increased, as shown by Fig. 1, to about $6 335 000, if capitalized

* Ex. Doc. No. 249, H. R. 53d Cong., 3d Sess., Jan. 1895.

Mr. Haupt.

at 4% on the average annual increment of $5 000 net. By this method the actual financial status of the utility is clearly set forth, as it shows the gross receipts from the capital invested, the cost of construction, maintenance, and operation; the influence of social, physical, and commercial agencies during the life of the enterprise and the average net returns, so that, under the doctrine of probabilities, the curve of future value at any given date may be closely approximated.

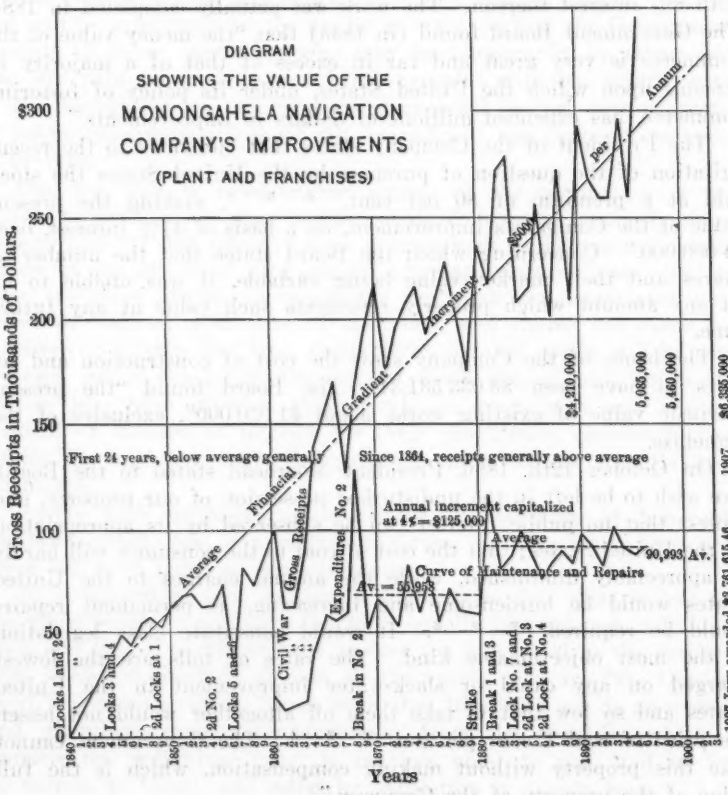

FIG. 1.

It eliminates, also, to a very great extent, the personal equation or erroneous judgment due to the desires of the parties at interest, either of buyer or seller, as it deals only with the facts as measured in dollars and cents and, therefore, should commend itself to all parties who may be retained to ascertain the "just compensation" which should always be paid when a private utility is to be acquired for public convenience.

Mr. Haupt.

In another instance, the Government has been negotiating for an important strategic canal route, of great utility, for more than 40 years concerning which the Chief of Engineers has reported that it would save $1 229 242 on the actual commerce now using it, and "probably considerably in excess of this amount", so that, to effect such an economy (regardless of risks and insurance), it would be justifiable to expend more than $30 000 000, if capitalized at 4%, and yet it hesitates to invest even one-third of this sum for the development of our domestic commerce. Surely an enterprise of such great utility should commend itself for immediate development.

Mr. Jewel.

L. L. JEWEL,* M. AM. SOC. C. E. (by letter).—This paper has been read with a great deal of interest and admiration. It is a masterly presentation of a difficult and complicated subject, and could have been made only after a close and unbiased study of the relations between private interests on the one hand and community interests on the other.

In studying the lines of evidence of value which are summarized on page 2413,† one is impressed with the unimportance, or rather, say, the indirect character, of some of them. For example, the market value of the securities of a property will very rarely, and may never, have any direct relation to its value. Even when there is a broad, free market for them, and the state of mind and of finances of the investing public is normal and undisturbed by any of the thousands of factors that may and do disturb it, the market value of the securities of a property will, to quote the author's apt phrase, reflect more "the hopes or fears for the morrow" than the value of to-day.

Again, the values of other properties similarly situated will seldom be available, and, when they are, could hardly be used except for purposes of comparison with a value already fixed by direct evidence.

Finally, it is hard to see how past cost necessarily has any relation whatever to present value. It is a measure of the efficiency—perhaps the honesty—of promotion, of construction, and of operation; but it certainly can be no dependable measure of present value, nor necessarily have any determinable relation to it.

It would seem, therefore, that the lines of evidence of value as given may be separated into two groups or classes: one containing the lines of direct evidence of value, and the other the lines of evidence which are always of secondary and often of no importance in determining value, but which, nevertheless, may be to some extent useful as a check or balance on the result obtained from direct sources. That is to say, the just value of a property for any purpose in which the consuming public is interested is directly dependent on two factors:

* Wytheville, Va.

† *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Jewel. (1) The cost of reproduction, starting with a naked idea and finishing with a going concern, as it is;

(2) What may be called a business estimate of value, depending on the many factors and conditions peculiar to that particular property and community as given by the author in the paragraph called "Fourth" on page 2413.*

A value thus determined could hardly be influenced, although it might be checked, by the other lines of evidence given.

Such a classification of the author's lines of evidence may be criticized as encroaching on the judicial function of weighing the evidence, and as being outside the scope of the paper. It seems to the writer, however, to be not only in harmony with the ideas expressed throughout the paper and with the foundation laid down by the law, but also to be of universal application, and therefore a fundamental principle of public utility valuation.

Mr. Dana. RICHARD T. DANA,† M. AM. SOC. C. E. (by letter).—The author, as he says himself, has assumed an ambitious task, and has produced a work that, if only for the emphasis so skilfully laid on the necessity for dependence on legal principles, cannot fail to further materially the development of scientific methods of appraisal work. The bibliography of legal decisions is timely and valuable. Perhaps People *ex rel.* Kings County Lighting Company *vs.* Wilcox *et al.*, and the Spokane Rate case, might profitably be added. The final paragraph of the paper, defining what the author calls a Golden Rule for the engineering appraiser in valuation work, is more than admirable.

Some of the definitions on pages 2392, 2393* are hardly as comprehensive as would seem to be necessary, in view of the fact that they are intended to establish a definitive basis for the development of principles heretofore somewhat obscure. Two, in particular, will be mentioned: "Property" and "Value". "That which belongs exclusively to an individual" is, in the writer's judgment, a rather inadequate description of "Property". Property connotes proprietorship, and proprietorship may be exercised by a corporation, a firm, a Government, or State, etc., each of which is distinctive, and may not be an individual. "The right of possession" seems to be hardly in the same generic class as the thing possessed, and is objectionable to the writer on that account. Webster's Dictionary recognizes both meanings of the term, but takes pains to separate them.

Economists have striven to define the term "value" satisfactorily, with but poor success to date. "A measure of the relation of two services growing out of the adjustment of mutual needs" implies, to the writer's mind, a good deal of abstruse theory. In its fundamental

* *Proceedings*, Am. Soc. C. E., for October, 1914.

† New York City.

concept, and postulating labor as the basic principle of wealth, this definition may be theoretically correct, but, for practical purposes in appraisal work, it seems to be too complicated. The other definition of this term, given by the author, seems to be hardly adequate, as the undesirable qualities certainly affect the value, and should be considered. Mr. Dana.

Any definition of value which presupposes a free market, a willing purchaser, a perfect and inflexible medium of exchange, is necessarily too restrictive. The science of economics needs for this term a better definition than any which has appeared thus far. Value is not cost, it is not price. For appraisal purposes, the value of a property might perhaps be called the quantity of its net assets, physical and non-physical, all expressed on the same basis of currency.

On page 2396* Mr. Alvord deprecates a "Jack of all Trades", particularly in appraisals, but he does not make it exactly clear, at least to the writer, just who ought to do most of the appraising, the "experts from several professions", or "engineering specialists from each type of utilities to be valued".

In every public utility corporation of considerable size there are usually several very able engineers who are thoroughly familiar with the property because of long connection with it and intimate association with its problems. These men, under the general direction of an engineer who has specialized in economics, costs, and appraisals, will make the best possible "team" for the solution of the composite problems that are inseparable from any appraisal, as has been proved many a time and oft. It has also been proved, to the tune of many millions of dollars, that the appraisal specialist need not be a specialist in electricity in order to make the best appraisal of an electric plant. Some of the worst appraisers of railroad property have been railroad men, some of the best painters have been poor judges of the value of pictures painted by others. To appraise an egg properly, one need not have been a hen.

Moreover, a specialist in economics, and of general engineering training, can find values in one kind of property to which he is led by knowledge of analogous values in another kind of property, which would be overlooked without such general experience.

Granted equal age and intelligence, the man who has had experience of appraising utilities of many kinds will make a better appraisal of any one kind than will he who has confined himself to that one kind of utility all his life, without opportunity to adapt the ideas gleaned from a wider field. This principle, indeed, is fundamental in all managerial processes, of which appraising is not the least.

* *Proceedings*, Am. Soc. C. E., for October, 1914.

Mr. Lavis.

F. LAVIS,* M. AM. SOC. C. E.—The Society and the Engineering Profession are to be congratulated on this valuable addition to the now somewhat voluminous literature on the subject of valuation. This discussion of the fundamental principles, and the clear definition of the points which can be defined, as well as the discussion of those about which there is still some doubt, will do a great deal toward placing the subject on a somewhat more rational basis than that on which it has hitherto seemed to rest.

The whole paper will well repay careful study, but there are some points which the speaker believes may be especially emphasized:

First.—That, although the reproduction method seems to be most generally applicable, there can be no absolutely hard-and-fast rules laid down to govern the methods to be used. It seems most important to point out that each case must be considered on its merits.

Second.—The necessity of collecting and properly arranging all the facts and data relating to each individual case before attempting to formulate opinions.

Third.—The six items cited as necessary to remember, in weighing evidence as to value, especially the first, referring to the fact that value can never be absolute, but is always a matter of intelligent opinion.

Fourth.—The very vivid description (on page 2416†) of the development process. Most particularly apropos being the reference to the "easy and virtuous hindsight" that tells us how the property might be reproduced by supermen, instead of causing us to remember some of our own very human lack of foresight which, to some extent at least, has increased, in some way, the cost of every piece of construction with which we have been connected.

Fifth.—The imperative necessity that engineers must be guided by the law as it exists and as it has been interpreted by the Courts. This, however, though eminently desirable, need not cause us to forget that both the law and its interpretation are subject to changes, and are sometimes at least viewed "in the light of reason" which we might possibly be able to make shine.

In the enumeration (on page 2413*) of those items which must be considered in order to arrive at the cost of reproduction, Mr. Alvord is very careful to point out specifically, in Item *D*, the necessity of estimating the depreciation, but he does not call attention to the equally important necessity of including also the appreciation. It is true, of course, that an intelligent estimate of the cost of reproduction in itself would cover the estimation of both appreciation and depreciation, and, therefore, both may be considered to be covered by Item

* New York City.

† *Proceedings*, Am. Soc. C. E., for October, 1914.

C; but as long as Item *D* was thought to be a necessary supplement to Item *C*, it would appear to be desirable also to include a further note in regard to appreciation, especially as this is so much more likely not to be fully allowed. Mr. Lavis.

There is probably a question as to how far engineers may go toward the determination of the complete value of any live operated property, especially of transportation systems. Mr. Alvord very appropriately points out the necessary co-operation of those experienced in the law, and in economics or finance, as well as engineering, in arriving at this final result. As far as the railways are concerned, it is equally necessary to obtain the aid of those skilled in operation, in traffic and its sources and destinations, and in other allied features, as they usually have a far more adequate idea than engineers as to the costs of development—a most important item of value.

The speaker is almost entirely in accord with the views expressed in the paper, and believes that, speaking generally, they apply equally well to all valuation. The paper, however, is distinctly written from the viewpoint of their particular application to the circumscribed and unified types of utilities, such as lighting and water supply, or even electric railway plants, rather than to steam railways, and this must be kept in mind. This is particularly noticeable in the statement that it is a "demonstrated fact that earnings from public utility investments do not fluctuate so greatly in financial depressions as do some other forms of investment." This most assuredly does not apply to steam railroads, but is generally true of utilities supplying individual municipal needs.

As far as these two general types of properties are concerned, there might be little difference in the principles governing their valuation were it not for the question of the use which it is proposed to make of such valuation when it is made. Valuations are made for many purposes, but the one which is of the most vital present interest is the use of this value for the purpose of the regulation of rates. It is when considering the values of properties for this purpose that the railway situation exhibits difficulties which do not seem to present themselves when considering isolated and complete units.

It is hardly necessary to quote individual cases or examples, as these will readily occur to those having experience in the railroad situation; it does seem worth while, however, to call attention to this point, and to the natural corollary, that therefore neither the market value of securities nor the past or present earnings can be used in the determination of values for this purpose.

Louis L. Tribus,* M. Am. Soc. C. E.—Many of the current articles on utility valuation give evidence of idiosyncrasies, but Mr. Alvord's paper should be considered as a very able, interesting, and comprehen- Mr. Tribus

* New York City.

Mr. Tribus.

sive presentation of rules and methods of arriving at values, together with much basic information and many common-sense comments.

In the speaker's earlier days, the then older engineers, trained largely in the "School of Hard Knocks", were looked up to admiringly, for from them emanated an atmosphere of good sense, and in their infrequent writings and discussions were found the epitomes of judgment, founded on sound principles, because they were the result of well-digested practical successes and failures.

Ofttimes, these same men could not, or would not, formulate the theories on which their works were built or deduce conclusions from them.

Engineers were but little bound by precedent, so that individual thought had fairly free rein.

This is not telling of the remote past, but facts prevailing within very recent years.

About 15 years ago, virtually began the great consolidation of allied interests and an awakening to the possibility of increased efficiency in management; valuation work as now known was then born, though only a crystallized phase of a principle prevailing from the days of Cain and Abel. Unfortunately, with the extreme swing of the pendulum, "efficiency" has become a word more nearly for ridicule than admiration, because many, even excellent, engineers have been led astray by the allurements of blank forms and colored inks, substituting a phantom for reality, and valuation as a newer hobby may easily reach the same goal.

Socialism is now the ruling doctrine of America, even if we do not fully acknowledge it, and unless level-headed men will interject common sense into public thought, there is no knowing to what extremes it may lead. The present public attitude toward public utilities is a marked example.

Engineers have been receiving an increasingly thorough technical training, though perhaps too much specialized at the start, and writers like Mr. Alvord have, sometimes with great clearness, made available for even the tyro, thorough discussions of subjects not found in textbooks and almost unknown to the teacher of a few years past. The teacher's good sense, however, often put into the mind of the student the broad principles which enabled him to make good selections of mental tools and meet the successive problems of practice with credit.

The socialistic idea, however, has already secured dominance in our larger cities, in the legislatures, and even in the Courts, with some phases to the advantage of the public, but not all.

This is nowhere more prominent than in the treatment of the public service corporations. Where is the group of capitalists willing to-day to invest in a new to-be-privately-owned water-works system? There is no safety in such ownership, with the restricted franchise,

Mr Tribus.

the low-rate, short-term hydrant contracts, subjected to the whims of constantly changing city officials, virtual confiscation of property at the end of the franchise period, and constant supervision and regulation of rates by public service commissions. Engineers are not wholly free from blame in producing that effect.

In the speaker's hearing, a well-known engineer objected to a certain city paying to a small water company any material sum for its plant, because "the city could not use it as a part of the larger system, which it would construct at some indefinite time, when it got ready." The company had not had a successful financial career, but had made possible the growth and development of the community served, bringing to the city, in increased taxes, far more than the interest on any sum that might have been required to purchase the plant. In many a valuation case the parties representing those in power have indicated a similar sentiment, though rarely so shortly or brutally expressed.

The horse trade referred to in the paper is the typical basis of all valuations for sale or acquisition of a public utility, except that, when a community is the purchaser, it wields an absolutely unfair advantage, for it can enter into ruinous competition and collect in the tax levy all losses, while the private owner goes steadily into bankruptcy, or accepts the offer made, whatever loss may be entailed.

Granted that arbitration of some form is resorted to, the owners are placed in the position of a defendant who has to prove every item of value rather than have his claims disproved by the community, consequently often many items of real value and legitimate expense are lost in the summations.

How are proper figures to be reached? Here comes the value of such theories as described by this paper and others somewhat similar.

Applying several of the methods will give several differing resultants; now for the horse trader's reasoning: The horse may live for so many years, he may encounter no accidents or illness, therefore he will be able to make good return on the purchase price; the would-be purchaser fixes $200 as the reasonable value, but might pay more, yet cannily offers $175. The seller thinks along the same lines, but remembers some veterinarian's bills, a large feed account, and some lost time, yet also has in mind some excellent earnings; he thinks of $275 with longing, but $225 rather than run risks, and asks $250. Finally, $225 is reached by dickering. No rule of Court based on comprehensive valuing would have brought that result, and thus is it with more important work. The expert evidence worked out on various bases produces an effect on the mind of the jury or Court, but judgment—or sometimes averages—determines the final decision.

If the arbitrators wish to convey the impression of great learning and very careful weighing of the evidence, a "dictum" accompanies the "opinion", and a "rule of Court" is born, to be quoted for all

Mr. Tribus.

time as a guide in other cases, to be followed, perchance, as sheep follow a leader.

Socialistic sentiment is at heart founded on absolute justice between man and man, but socialistic action and legislation is more the result of the one-time-oppressed squaring accounts (and a little more) with the oppressor.

The public utility operator of the past, as a class, has not served the public with strict justice and consideration, and so is not entirely free from responsibility for the judgment often meted out to-day.

All the more reason why the engineer called into a valuation case shall be fair in his advice, as well as intelligent and comprehensive in his study.

That brings us to the point that is too often overlooked, no two cases, generally speaking, are alike or will bear the imposition of the same rules.

It is incumbent, therefore, for the engineer to make a study of the local conditions that affect the utility, its origin and financing, its construction, its operations, its effect on the community, its financial rewards or, perchance, losses, its obsolescence, whether due to poor original judgment, or to radical improvements in machinery, methods, etc., in fact, all the many factors which enter into success or failure.

Then, applying such methods as seem to have applicability, figures will be derived that can have some force in determining the final "horse-trade" judgment.

Without following certain illustrious examples, in condemning the Courts, quietly, engineers may remember that at one time many decisions happened to favor those in litigation who were represented by particular lawyers retained by the party having the longest purse, but to-day votes are more apt to prevail, the decision going to interests wielding the balance of power.

Sometimes non-residence ownership enters the problem, not that the engineer should consider it a factor, but to look out for it and forestall injustice by the thoroughness of his presentation of facts and deductions.

In this discussion the engineer is considered as a judge, not as an advocate; of course he may advise counsel as to points favorable to his side, but on the stand he must be absolutely impartial; and a righteous judge will give great weight to the testimony of a just engineer. We would not intimate, either, that the unrighteous judge is in the majority; something in the nature of such a trust brings out the fair qualities of a man, if they are in him, but too often, influence is stronger than right.

Valuations for rate-making, and taxation, are simply different phases of the same problem, only giving to a possibly injured owner a little greater chance for later redress through review.

Mr. Tribus.

The one point that the speaker wishes to emphasize strongly is the futility of rules for reaching decision as to value, though their benefit is unquestioned for suggesting a range of values, which may be somewhere near the truth; more important than all, however, is a thorough study of all the different factors in light of prior experience as to the financing, designing, constructing, and operating problems involved in the utility under discussion.

The engineer without practical experience is as valueless in handling such questions as the lawyer would be in designing the utility itself.

Let us have less of precedent, fewer differentiations of rules and dicta, and more horse-sense in valuation work.

Mr. Mayer.

JOSEPH MAYER,* M. AM. SOC. C. E. (by letter).—This admirable paper clears up many of the confusions pervading the discussion of the valuation of public utilities but leaves some points still in a fog which arises mainly from the author's assumption that law and the decisions of Courts, and not justice and the nature of things, are the supreme authority in valuation. Law is but an imperfectly expressed authoritative opinion of the legislature attempting to define justice, and must be and is construed so as to produce justice or such relations among men as will secure the public welfare. The public welfare, and not the Constitution, is the supreme law of every land, as the Romans, the creators of the first great system of laws, already knew.

The author is aware that the idea and nature of value is the foundation of all valuation, but refrains from giving an explicit definition of the idea of value that forms the foundation of his paper. Instead, he gives us a miscellaneous assortment of legal and economic contradictory definitions of value, none of which is quite adequate as a basis for the discussion.

He gives us:

"*Value.*—The sum of the desirable qualities which render a thing useful and sought, measured in money; a measure of the relation of two services growing out of the adjustment of mutual needs."

These are two contradictory definitions. The sum of the desirable qualities which render a thing useful and sought cannot be measured in money because the amount of money which such a sum commands depends on other independent factors. Air is essential to life, and therefore presents a very large sum of desirable qualities, but its value measured in money does not measure this sum. The second part of this definition, which contradicts the first, is more nearly true, because value is a measure of relations between many services, no two of which define value. It is quite indefinite, however, because there are many different relations between services. The value of a thing

* Montreal, Que., Canada.

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or service is a measure of the quantities of many other things it will buy. Money is one of these other things. The quantity of money which can be obtained for a thing is its price. Money at a given time and place has a definite value which can be more easily ascertained than the value of most other things, and its value is more constant in time and space than that of most other things. Therefore, it is the most convenient approximate measure of value. The value of money, however, sometimes fluctuates rapidly; the price of things becomes then a very unreliable measure of their value.

The author also gives us:

"*Market Value.*—The adjustment of two services, *i. e.*, as between a willing seller and a willing buyer under open conditions of competition."

This defines the ratio of the values of the two services which would prevail if competition were universal, but it does not define the competitive value of either; many comparisons are needed to define any kind of value of anything. Since monopolies are spreading, the prices and values which co-exist with monopolies are of most practical importance. If the clause "under open conditions of competition" means simply with competition between many independents, not co-operating buyers and sellers of the two services, then the definition gives the ratio of their values under the given conditions. This ratio it is possible to ascertain if the described conditions actually exist.

Another:

"The sum of money a thing will produce to the seller when sold."

This is the price, not the value.

Another:

"The quantity of labor and capital necessary to produce a given article, or, in other words, the actual cost of its production, is the true criterion of its worth."

This is a clumsy attempt to define the cost of production, not the value. The price of the labor and the interest on the capital used in the production of a thing, together with the profit of the owners and the rent of the land used, with the taxes and other incidental expenses, determine the cost of production, but not the value of a thing which can be produced. Many valuable things, like land and genuine antiquities, cannot now be produced, and therefore have no present cost of production, but they have value.

The author does not state which of these contradictory definitions he believes to be true, and, throughout his paper, does not adhere strictly to any one of them. He has in his mind a much better definition of value, resulting from his wide experience in valuation, which is more nearly correct than any one of those he gives, to which

he generally adheres, and which creates the great value of his paper, but he sometimes abandons it in deference to the Courts, and then he becomes confused and obscure. Mr. Mayer.

The Courts are not authorities on logic. The principles of logic are above the Courts, and demand that the realities must govern all definitions. There are groups of things and groups of phenomena in the external nature; the things or phenomena in each group have certain features in common. These groups are separate from each other; there are few or no transitional forms or phenomena between them. The definitions must draw the lines where Nature has drawn them. True definitions can only be given by those who know thoroughly the things or phenomena they attempt to define. Economists, and not Courts, are most competent to give the true definitions of value which must serve as the basis for valuations. Courts, however, are competent to construe the laws and to decide which values in the possession of individuals can, and which cannot, be taken by the public without compensation. The attempts of the Courts to define values have mostly the purpose to distinguish between these two varieties. It is quite proper to define different varieties of values, but the definitions should describe groups of realities which actually exist in the external world, not only in the mind of the reasoner, and which have real limits that separate them from other groups; and nothing should be called a variety of value which does not belong to the species value.

Court definitions which describe actually existing varieties of value are practically important for valuators, and must be considered in their proper place. The Courts mention fair and reasonable value and fair and reasonable return on the same, without giving definitions of these terms. Their meaning must be inferred by studying the decisions based on them. Until they have been so defined as properly to describe actually existing groups of value which have definite limits separating them from other groups, they cannot be used as the basis of a logical discussion. Beside the nature of value, the laws which govern the changes in values are of the utmost importance in valuation.

Man desires to obtain certain things; he will produce them, if this is the easiest way to get them, and if he likes them more than he dislikes the trouble of producing them. For some things he will make great, for others only small, sacrifices or efforts. The amount of effort or sacrifice he is willing to make, if necessary, to obtain a thing, measures for him its apparent usefulness, and this is the upper limit of the value of the thing for him. The actual value of the thing, as long as he lives alone, is the effort he must make to get it; this is also its cost of production. Things of the same value, under these conditions, have the same cost of production, and things of the same

Mr. Mayer. cost of production have the same value, provided their apparent usefulness is at least equal to their cost of production. The usefulness does not measure value, it only fixes an upper limit which value cannot exceed.

The usefulness may also be defined as the degree in which a thing serves the needs of life. It is, however, not the actual but the apparent usefulness which is measured by the amount of effort one is willing to make to obtain a thing. Things like land, which cannot be produced, can be acquired by effort, and the necessary effort to acquire them measures their value if this does not exceed their usefulness. These are the laws of value, only as long as a man is alone. Other laws prevail where many different men live together and exchange things with each other. If all men were exactly alike and had the same possessions and surroundings there would be no reason for exchanges, and the laws of value above described would also hold for a society of men.

In a society of different men with different possessions and surroundings, the cost of production of the same thing is different for different men. Each will now tend to produce those things or services for which he is best fitted, and will exchange his surplus of these things or services with others to obtain those which he can only produce with great effort or not at all, to the profit of all.

Markets then arise where the prices of things are fixed so that the efficient demand for them equals the supply. The efficient demand for a thing at a given price is the quantity asked for by those willing to pay the price. The supply is the quantity offered. Both change with the price.

The supply of such things as cannot be produced is necessarily not governed by their cost of production, but, if they are owned by many who do not co-operate, it is governed by the prices and by their usefulness to their present owners. If all the owners co-operate, such a quantity only may be put on the market as will bring the largest total price. A small quantity may then bring a larger price than a large one. Where all or most of the producers of a thing co-operate to limit the supply, higher prices are secured than with free competition among many independent producers.

The supply of such things as are produced by many independent, not co-operating, producers, tends to increase when the average profits of their producers are larger than in other productive industries, because capital will flow into the most profitable channels.

Competition is disappearing in many lines of production, and especially so in public utilities. With the disappearance of competition, other laws begin to govern prices and profits.

Without regulation, the prices of the products of monopolized industries are much higher than would be required to secure the same

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average profits as in competitive industries. The difference between monopolized and competitive profits goes largely to promoters and reorganizers of monopolies. Capital invested in unregulated monopolies is generally highly profitable. The promoters and reorganizers obtain the required capital from the public by selling stocks and bonds at prices which bring to the buyers often only about competitive profits. If the consumers of monopolized products organize and fix such prices for them as will give but competitive profits, they can get the required capital from the public, and the prices will be the same as if competition existed. The purpose of the regulation of monopolies is to obtain monopolized products at competitive prices. There are two great difficulties to be overcome in such regulation. The first is to avoid injustice to the present owners of the stocks and bonds of monopolies; the second is that of securing efficiency of management. Valuation is mainly intended to overcome the first; the second, thus far, has been but little considered.

The Courts endeavor to do justice to the owners and the public by giving to the owners a fair return on the present fair value of their property.

The author endeavors to ascertain this fair value of the property of monopolies, and consults for this purpose the Fifth and Fourteenth Amendments of the Constitution of the United States and the decisions of the higher Courts.

The two Amendments prescribe that no State shall deprive any person of life, liberty, or property, without due process of law, and that private property shall not be taken for public use without just compensation. To grasp the meaning of these clauses we must know what is due process of law, in taking private property, and what is just compensation. This we can find by observing closely how private property is taken for the State or Nation. The legislatures are constantly engaged in changing values and thereby taking and giving property to private persons. Legislation might be defined as a process of changing values so as to secure the public welfare thereby. When a tariff law is passed, the values of many goods extensively imported are changed, and with them the values of the factories making them. If States introduce prohibition, the values of breweries and distilleries in these, and to some extent in other, States, are reduced. If a State fixes minimum rates of wages, it changes the value of labor. All tax laws change values and take property. No compensation is ever given by the Courts. It is evident that the Courts either consider these values not to be property, or they believe that no compensation is here just compensation, or that a manufacturer knows, when he enters a protected industry, that the protection may be removed by the legislature and that the value of his plant and the income from it is influenced by this knowledge to such an extent that he receives thereby

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compensation for the risk. This, however, introduces an element of gambling into such industries, which is objectionable. The Courts would probably not sustain this if it were practicable to remove it. These instances are given to show that it does not follow from the Constitution that compensation will always be given where private property, or at least a part of its value, is taken by the State or Nation. They show that the public welfare and the nature of things are the supreme law. The security of property from aggressions by the State, therefore, is not absolute unless all these values are not considered property. It is not important for our purpose whether the values which we consider are or are not property. What is important to know is, which values can and which cannot be taken by the State without compensation. The Constitution gives us no answer to the question. Therefore we must go to the decisions of the Courts in similar cases, and to the principles implied if not expressed in these decisions.

The author has endeavored to find that value which the Courts will award to the public utility companies, and he gives the method he follows in ascertaining it. The variety of value awarded to these companies must be a group having a real existence, and it must be within the species value. All kinds of value are defined as the quantities of other services or things a service or thing will buy; this defines the species value.

Under different conditions, any kind of service will have different values. Varieties of values can only be defined by defining the conditions which influence their value. One variety is that value which exists for the lonesome producer. Another variety exists where there is universal competition, another where unregulated monopolies exist, and many others with regulations of various kinds. The most important condition influencing the value of regulated monopolies is the kind of regulation adopted. The Courts insist that the present value is the fair value to be ascertained. The leading lines of evidence to ascertain the present value are, according to the author, a statement of the earnings and expenses during recent years; the market values of the securities of a property where a broad and representative market exists; the cost of reproduction less depreciation, past cost, and other factors. The Courts and the author neglect to mention the most important factor influencing the present value of a public utility, namely, the kind of regulation to which it is to be submitted in the near future. They are attempting to determine the present value of these utilities as if it were independent of the regulation to be adopted, they practically assert that the value can be determined without knowing what regulation will be adopted. This is a mere legal fiction, in the minds of the author and the judges, which contradicts the facts. There exists no variety or kind of value which

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can be ascertained without knowing all the influential conditions which determine it. To illustrate: The value of an old house is found mainly by estimating the future net revenue which will be obtained by the owner, and estimating the present value of this estimated net revenue; if, however, the value at a near-by future date of the then future net revenue differs largely from the present value of the future net revenue, the former has an influence on the present intrinsic value. The intrinsic value of a house is that value which would prevail if the buyers and sellers would all correctly estimate this future net revenue and the present market value of it, and if they were all governed in their buying and selling by these correct estimates, and by similar estimates for near-by future dates. This intrinsic value the valuator tries to find. It is a definite thing at any time, it does not change rapidly, though the value shown by actual sales may fluctuate widely above and below the intrinsic value. The actual buyers and sellers are governed by more or less erroneous estimates of the future net revenue, based principally on the present net revenue and a consideration of the causes which will change it in the future; they may also consider causes which change the prevailing rate of interest. If the interest rate is rising, the present value of future payments decreases; this reduces the value of the house in the future, other things remaining the same. The knowledge that the value is increasing or decreasing also influences the present value. The buyers and sellers will also consider the ratio between values and net revenues prevailing in the same neighborhood for similar property; they find an approximately constant ratio between the two and then multiply the estimated net revenue by this ratio to obtain the value. If the house considered is out of style and out of harmony with the demands made by the possible buyers of such houses, he will not try to find the cost of production of a similar new house and the depreciation by obsolescence. He will estimate, from the present net revenue and the causes of change, the future net revenue and its present value; or he may ascertain the cost of production (including the cost of land) of a new house adapted to present tastes which gives the same present value of the future net revenue.

To make this clearer, consider the value of a large electric power-plant of, say, 100 000 h.p., with reciprocating steam engines. Such plants are now built much more economically with steam turbines. The value of the old plant is the present value of its future net revenue. The future net revenue depends largely on the future price of power. The price of power, under competitive conditions and without regulation, is mainly governed by its cost of production by the most economical methods known. The cost of a steam turbine power-plant which can obtain the same future net revenue as the old plant with reciprocating engines is approximately the value of this

Mr. Mayer. old plant, because plants giving the same revenue have the same value, and the value of a modern plant is, under competitive conditions, approximately equal to its cost of production. It would be most absurd to attempt to find the depreciated value of the old plant by finding first the cost of reproduction new of a similar plant and then the difference between this cost and the value of the old plant. The similar new plant would be antiquated, and its cost of production does not indicate its value. Its value could only be found by ascertaining the cost of production of a modern plant bringing a future net revenue of the same value. After this the values of the antiquated new and the antiquated old plant would have to be compared by comparing the present values of their future net revenues. Since the cost of reproduction new has no influence on the value of an antiquated plant, it is a waste of time and money to determine it. The cost of reproduction new of an old antiquated plant, therefore, is absolutely worthless as an evidence of value. In all cases where obsolescence is an important factor, the only rational method of finding the value is by comparison with the cost of production of an equally valuable modern plant, or one producing a future net revenue of equal present value. Any cost of production, however, is only under competitive conditions an approximate measure of value. Where monopolies prevail, the cost of production does not measure the value. With regulated monopolies, the value depends mainly on the kind of regulation adopted. As the author states, the value of a well designed plant meeting the needs of a growing market is at least equal to the cost of production under any kind of practicable regulation; but the value may be much more with the existing methods of regulation. The recent net earnings are only a leading evidence of value when they indicate the probable future net earnings; this they do only if the regulation in the future is the same in its effect on the value as that in the recent past.

The author in his method of valuation practically assumes that this regulation will be the same in its effects on value as the regulation of the recent past; therefore he believes that the fair value of a public utility is that value which exists if the future regulation produces the same values of the present properties as that in force in the recent past; and he consistently follows this definition, except where he is compelled by deference to the Courts, for the purpose of obtaining their approval for his valuations, to deviate from it. As the author states, the Courts are very cautious, and cannot be induced to commit themselves to wide generalization; this is the function of the legislatures, not of the Courts, and the latter should give a clear definition of the fair value of utility companies; then they could be valued by ordinary business methods of valuation without insisting on the

exact ascertainment of the value of the carcass for the purpose of obtaining the value of a live horse. Mr. Mayer.

The author says that valuation consists in collecting the facts influencing value and in judging them. He is practically compelled to omit the most important fact—the future regulation—and he can only describe superficially the various operations to be performed in the judging, but he cannot find a real method of judging without first, implicitly, if not expressly, assuming what kind of value he desires to ascertain. A clear idea of what you want to ascertain is absolutely necessary for finding the methods of ascertaining it. The present value of public utilities is very nearly identical with the value which would prevail if the future regulation were the same as the present one. Substantial justice, or as close an approach to justice as practicable, would be done to the present owners of the securities of these corporations if their value were ascertained with this assumption, and if a fair return on this value would be paid to the stockholders. This, however, must not be decided by the sense of justice of any one, or any class of individuals, but by the will of the whole community expressed by the legislatures or the Courts. Most of the conflict of opinion in the discussion of valuation arises from conflicting ideas of justice. With universal competition, fair prices and values rise, if each producer is left free to offer his products at whatever price he pleases. Under such conditions the average profits in all industries tend toward equality, therefore the idea arose that justice requires that each should be free to choose the prices at which he is willing to sell or buy. After natural and artificial monopolies became numerous and important, this absolute freedom of action did not any more produce just prices; the old ideas of justice, however, were for a long time enforced by the Courts and legislatures, and great injustice in the distribution of income and wealth resulted therefrom. This brought about a conviction that such absolute freedom cannot be allowed, and efforts were made by the legislatures, the Courts and the, for this purpose created, public utility commissions, to establish just prices. Under the old laws very large returns on the invested capital were secured in the monopolized industries, and their value, therefore, often largely exceeded their cost of production. Whether the values of monopolies created under the mixture of freedom and regulation of the recent past shall belong to the owners of the corporations or partly to the public is still unsettled. Similar changes of conditions and of prevailing ideas of justice characterize all historical development, as is shown by Henry Sumner Maine.* He says, in his chapter entitled "Legal Fictions":

"In progressive societies social necessities and social opinion are always more or less in advance of law. We may come indefinitely

* "Ancient Laws."

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near to the closing of the gap between them, but it has a constant tendency to reopen. Law is stable and some societies progress. The greater or less happiness of a people depends on the degree of promptitude with which the gulf is narrowed."

Much of the difficulty in valuation arises from the insistence of the Courts on their legal fiction that it is possible to find the value of a regulated public utility company without knowing the nature of the regulation. This necessitates the abandonment of the only correct and reasonably accurate business men's method of finding the value of a public utility company by estimating the present value of its future net earnings; and forces the adoption of some artificial and extremely complicated substitute that pretends to find its value without considering the regulation, and that is nevertheless compelled to assume implicitly a definite kind of regulation, as it is quite impossible to solve the problem in any other way. No agreement on questions of valuation is possible until either the Courts change their course or the legislatures settle the question by statute law.

Another important legal term requiring a clear definition is "a fair return". For the stockholder of public utility companies, this is just as important as the meaning of "fair value", unless he is offered cash payment for the fair value of his property. It makes an enormous difference whether 5 or 8% is considered a fair return, and this should not be left to individual judgment, or merely guessed at, but should be determined on such known general principles that the investors can foresee the decision. There is substantial agreement, however, that a fair return for any monopolized industry is the same return on its value as is obtained in competitive industries, or a return just large enough to obtain the necessary capital; but there are individualists and socialists defending two radically different methods of distributing the profits of a monopolized industry among the different enterprises engaged in it. The individualists say the regulation should be such that the profits in every enterprise will be the same as they would be if competition existed. With free competition the profit in every individual enterprise is different; some bring large dividends, some go into receivers' hands, some are entirely abandoned on account of changes in the arts, but the average rate of profit in every industry at the same place and time tends to be the same. There are differences of average profit at different times and places. In a very new country, profits are larger than in the East. The average rates of profit at different times and places vary mainly with the rate of interest, and they are often larger in growing than in decaying industries. To bring about such a condition of things in the monopolized industries, it would be necessary to ascertain the average rate of profit in competitive industries and to establish such prices of monopolized products as will produce in each monopolized industry the

same average rate of profit as in the competitive industries at the same time and place, with slight variations to bring about the needed increase or decrease of supply. The prices of gas and electricity, the street car fares, etc., would have to be different in different places. The differences would have to be equal to the differences in cost of production with ordinary methods of production, as is approximately the case in competitive industries. The profits of the different enterprises would then depend mainly on the efficiency of management, or the amount of intelligence, conscience, and energy put into each enterprise, and the same inducements to secure efficiency would exist as with free competition. Mr. Mayer.

The writer has roughly sketched such a regulation in a previous paper.*

The public utility companies may endeavor in their regulations to approach this ideal of different profits in proportion to efficiency of management, or they may set before themselves another ideal, namely, equal profits in all enterprises, large enough to secure the necessary capital for increases of plant. If this latter ideal were followed consistently the stockholder would have no pecuniary inducement to select efficient directors, and the directors chosen would have none to obtain efficient management; or to adopt the most judicious salaries and wages for all the employees and to employ the best men obtainable for them; or to watch the performance of every one and to advance them for merit only; or to study all proposed improvements and to adopt them quickly if meritorious. All these things would have to be obtained through the supervision of the public utility commissions. The directors would become useless puppets, and they would soon be discarded as an unnecessary incumbrance. If the ideal were followed consistently, the stockholders would become practically bondholders, and money could be obtained at a moderate rate of interest, either by the sale of nominal stock or of bonds. The risk would be carried by the public, and the management would become entirely responsible to the public instead of to the stockholders. The public would become the virtual proprietor at the moment the ideal was adopted and consistently followed. Good results can be obtained by public ownership with public management, but not by public ownership and risk with private management.

This ideal of regulation is not yet followed consistently by the Courts and the public utility commissions, but they are gradually being compelled to adopt it by the manner of fixing the prices of monopolized products.

The consumers or one or more of the utility companies complain that the prices are too high or too low. A Court or a public utility

* "The Just Value of Monopolies, and the Regulation of the Prices of Their Products", *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 455.

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commission is requested to fix fair prices. They proceed to value the property by some extremely artificial and therefore inaccurate method, which must hide the fact that some kind of regulation is necessarily assumed in the valuation. After the valuation is obtained, a rate of profit is fixed which is believed to be just adequate to secure the necessary capital for extensions, and the prices are adjusted to produce this rate of profit. The risks of investments in controlled public utilities are peculiar and entirely different from those in competitive enterprises. Therefore it is very uncertain what rate of profit will make it possible to obtain the necessary capital for extensions. The Courts generally let a master decide this knotty question; the public utility companies guess at it themselves, as this is the best they can do under the circumstances. To fix a just rate of return for average efficiency of management, it would be necessary to ascertain the average rate of return in competitive enterprises and to create such conditions for the monopolized industry that the risks of capital are the same as with free competition. To do this to settle an individual dispute is impossible. An intelligent guess is therefore the best practicable solution of the problem. The commission or Court would undoubtedly fix a lower rate of profit where they find legal evidence of gross inefficiency or dishonesty, or might reduce grossly excessive salaries, but, for most of the factors which really govern the efficiency of management, no legal evidence can be presented to the Courts or commissions, and therefore they cannot be considered; but, even where such factors are legally known, it is impossible by the available machinery to calculate or estimate correctly the amount of change in cost of production caused by them and to allow for it by a different rate of profit.

The purely mental qualities of intelligence, honesty, skill, and energy, which cause such large differences in the cost of production of the same goods, selling at the same prices in competitive industries, and consequent large differences in profit, cannot be directly measured; they can only be judged by the material effects they produce. The Courts and commissions, by the now prevalent methods of fixing prices of monopolized products, are practically compelled to ignore all differences in such mental qualities which cannot be measured by their method of procedure, and to award equal profits to all enterprises, or equal compensation to intelligence, honesty, skill, and energy, and to stupidity, dishonesty, incompetence, and laziness. The Courts are established to settle individual disputes, and they cannot deviate from their usual method of considering each dispute separately from all the others. The utility commissions must either abandon this method, by creating such just systems of prices as would prevail if free and fair competition did exist; or they must obtain the capital for monopolized industries at a fixed and consequently low rate of

interest; and they must manage these enterprises themselves, discarding the private boards of directors elected by stockholders. Until one of these alternatives is chosen, increasing inefficiency and expensive capital with consequent high prices are inevitable. Statute law must decide which of these alternatives should be chosen for the different monopolized industries. Mr. Mayer.

PHILIP BURGESS,* M. AM. SOC. C. E. (by letter).—The writer has read this paper with great interest. It is a very timely and valuable contribution to the literature on the subject, and will be appreciated by any one interested in the matter, and especially by any one who has had occasion to follow through a particular case, as developed by hearings before a Court or commission. That members of the Engineering Profession frequently do not have a clear conception of the fundamental principles involved in the subject is very apparent from their testimony before Courts and from their discussions of papers, as printed in the publications of this Society. Such differences of opinion can be accounted for only by the fact that engineers have not taken the time, or have not had the ability, or opportunity, to acquaint themselves with the present status of the subject of the valuation of utility properties, as determined by decisions of the Courts. It is apparent, moreover, in many instances, as indicated by Mr. Alvord, that engineers frequently appear to know more about the subject than do the Courts, a position which, of course, leaves the witnesses open to severe criticism, or even to ridicule, and has a tendency to lessen the value of all expert testimony, even when given by qualified witnesses. This has undoubtedly called forth the frequently cited remark by a Court to the effect that expert testimony is the most unsatisfactory of all evidence. Fortunately, this criticism is not confined to engineers. Mr. Burgess.

The writer has had the pleasure of listening to Mr. Alvord's testimony when on the witness stand for several days, and has been much interested and impressed by the fact that he always has had a clear conception of the matter at hand, as he saw it, and has always started with some fundamental conception of the principle involved, and then has endeavored to follow this conception through to a logical conclusion. It is undoubtedly true that there is no subject before the engineer to-day which requires so clear a conception of the fundamental principle involved as does the appraisal of properties devoted to public use.

One instance may be cited to show how much the value of the testimony of an engineer may be weakened unless he has a clear conception of these principles. In a water-rate case, in which the writer was interested, a certain witness for the utility—who is a member of this Society—submitted a report on the cost of reproduction

* Columbus, Ohio.

Mr. Burgess.

new of the properties in question, and, when the report was handed in as evidence, stated that he did not know whether or not the cost of cutting through pavements over pipe lines should be included, and that such cost was not included in the report. Subsequently, Mr. Alvord, also called in by the company as a witness, stated why, in his opinion, the cost of cutting through pavements should be included. Later, the first witness asked to be allowed to state that he now believed that the cost of cutting through pavements should be included.

Under such circumstances, one can but ask the question as to whether or not the witness would have made up his mind on such an important item so quickly had he been on the other side of the table. It is very apparent that the witness did not have a clear conception of the fundamental principles involved, or that he was prejudiced.

Such instances of disputed items, whereon, perhaps, the Courts have not given deciding opinions, arise in all cases of valuation. It seems to the writer that, unless the engineer wants to appear in an unfavorable light, he must have a logical reason for arriving at his conclusions in all matters pertaining to the case, or he must state that "he does not know". He may well qualify such a statement to the effect that the matter is a legal and not an engineering question, and in fact is one for the Court itself to determine. If the Court decides that the item in question is a proper one to include, the amount may be stated to be so much. The point as to whether or not the item is a fair one to include may far better be argued by the attorneys in the case than by the engineering witness.

Doubtless one of the most valuable features of Mr. Alvord's paper will be found to be the legal citations and references with which, of course, any engineer who pretends to qualify for appraisement work should be familiar.

Mr. Alvord has indicated, in characteristically clear language, the four fundamental lines of evidence to be considered in making an appraisement. These are: (1) the cost of reproduction new; (2) the market value of securities of the company; (3) the comparative value of similar properties; and (4) the past costs of the properties in question. The writer believes that he is warranted in assuming that the importance of each of these items, in the author's opinion, is indicated by the order in which they are placed. In fact, in two instances in water-rate cases wherein the author and the writer have been jointly interested, although on opposite sides, the author has been contented to adopt his conclusion, as reached by investigations of the cost of reproduction new, without any apparent investigation into the actual past cost of the properties or of many items comprising them.

Mr. Alvord's paper may be considered as a very able argument, or demonstration, of the value of the cost of reproduction new as the foremost line of evidence to be considered in arriving at the value of public utility properties, especially if we include his qualifications of such costs new as indicated in his fourth line of evidence to be considered, namely, the relations of the utility to the consumer, the value of the service to the consumer, and the general business status of the properties. Mr. Burgess.

It would also seem to be apparent that Mr. Alvord is very much disappointed that some appraisers have seen fit to give so much value to such evidence as past cost of the properties. As the author very well states: "What does it profit us to discuss the details when this serious and fundamental difference exists?" The writer believes that, after all, except in the case of extremists, there is not so much difference of opinion among appraisers who are well informed in the matter as would at first appear.

As stated in the paper:

"The function of valuing a public utility property consists essentially of two distinct operations:

First.—The collection of all facts and evidence tending to throw light on the question of value."

It cannot be disputed that the best evidence as to the cost of the many items composing the properties in question is to be found in the books of the company. It is not denied that the conditions of construction may be different to-day from what they were when the plant was constructed. Nevertheless, the actual past costs, especially of particular or individual component parts of the plant, are facts, and are always of interest and value to Courts and appraisers. Such facts may very well be considered by appraisers, even in developing evidence or opinions along the lines of reproduction new, as of to-day. In this connection, it is necessary to consider what are the apparent differences between the conditions under which construction must take place to-day, as compared with those under which construction actually took place in the past; also, how such conditions affect costs.

As a matter of fact, it seems to the writer that the cost of reproduction new method of valuing properties is largely, if not wholly, a comparative method. There is, of course, no intention of actually constructing new properties exactly like those in question, but the appraiser must consider all evidence which may throw light on the probable sum that would be required to construct such a new comparative plant. A determination of the cost of reproduction new is essentially a matter of detail, comprising determinations of the probable cost of reproducing new many items, or component parts, of the comparative plant. Although it is true that comparative costs of other properties or units and estimated costs new of the

Mr. Burgess.

comparative plant by contractors or by engineers are all valuable aids to the appraiser in arriving at the probable cost of reproduction, it seems to the writer that unquestionably the actual past costs of the properties or units in question, as shown on the books of the company, are the best evidence for the appraiser by which to estimate the cost of reproduction new, provided, of course, that he considers and allows for all comparative conditions of construction between the past and the present.

Doubtless this is why appraisers, in many instances, have considered the past costs of new properties to be the best evidence as to their present value, because, of course, it is recognized that, in comparatively new properties, the conditions of construction under which the so-called comparative plant may be built are similar to those under which the properties in question actually were constructed. With older properties, however, this is not generally true, and it is on the older properties that there will be greater differences of opinion as to present value on account of this difference between the past cost method and that of reproduction new.

There are, however, certain very material differences in the appraisal values which may be obtained by using the past cost and the cost new as of to-day. These apply both to tangible and intangible costs.

Among the costs of tangible property included in reproduction and not in past costs are such items as the costs of cutting through pavements over mains and services—pavements which were constructed subsequent to the laying of the pipes. Such costs do not appear on the books of the company, but, if we are to reproduce the existing properties as of to-day and under present conditions, the costs of cutting through existing pavements is an element of the cost of reproducing the properties as of to-day. In the same way, in estimating preliminary costs, such as engineering, superintendence, interest during construction, and others, there will also be a difference between the amounts estimated for the cost of reproducing the properties and that actually incurred during the construction of the existing plant. Preliminary costs which may be entailed by the construction new of the comparative plant are purely matters of opinion based on facts and the knowledge of the appraiser as to such comparative costs of similar properties. They are estimated as an aliquot part of the complete plant as of to-day. The actual preliminary costs incident to the construction of the existing properties appear on the books of the company, and, of course, are a similar part, or proportion, but of a much less valuable property. Consequently, preliminary costs based on past costs almost invariably will be less than preliminary costs as estimated for the construction new of the comparative plant. This is because such overhead costs as engineering and general supervision

frequently are paid for entirely out of earnings during recent years and do not appear in the plant account. Mr. Burgess.

In the same way, there is frequently a marked difference between the results obtained by considering the two lines of evidence in determining intangible values, such as going value. The past cost method requires consideration of the actual revenues and operating expenses entailed by the plant during its early history. By some appraisers, this method is carried further to a consideration of whether or not early deficits have been compensated by subsequent earnings of the properties. By the comparative method, however, it is necessary to consider the earnings and expenses of the existing plant during recent years and to project these into the future with a view of determining the probable receipts and expenditures of the comparative plant, the construction of which is conceived to begin on the date of making the appraisal. Here again we have two lines of evidence, the one based on past costs, the other on the future, or present cost of developing the business of the so-called comparative plant. It is only an accident if they agree as to the result. Of course, going value, under the comparative or reproduction method, never can be negative, or compensated. So long as the properties are earning they have a going value.

On account of the many past appreciations and depreciations of the several component parts comprising the properties, none of which changes in value appears on the books of the company, but all of which do enter into the cost of reproduction new as of to-day, it is apparent that, in most instances, there will be considerable differences in the net results as obtained by using past costs and costs of reproduction new in estimating the value of utility properties, especially in the case of utilities which have been rendering service for comparatively long periods of time. As previously stated, however, it is the writer's belief that there need not be very great differences of opinion among appraisers as to the net results obtained in valuing a particular property, especially if we recognize the fact that the cost of reproduction new, though a most valuable line of evidence, is not the only line to be considered. That it is not the only line of evidence to be considered is well indicated by the Supreme Court of the United States in the Minnesota Rate Case, as follows:

"The cost of reproduction method is of service in ascertaining the present value of the plant, when it is reasonably applied and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture. It is fundamental that the judicial power to declare legislative action invalid upon constitutional grounds is to be exercised only in clear cases. The constitutional invalidity must be manifest, and if it rests upon disputed questions of fact the invalidating facts must be proved. And this is true of asserted value as of other facts."

Mr. Burgess.

In the writer's opinion, it is difficult to see how any appraiser can have at hand more relevant facts than the figures which appear on the books of the company as indicating actual past costs, especially of new properties. It must be considered that, in any particular instance, the only additional facts available are the costs of other properties generally similar in character to those in question, but, of course, not identical with them. It is the writer's belief, therefore, that, after all, there is not so very much difference of opinion among well informed appraisers as would at first appear. The cost of reproduction new is of very great value, but is not by any means the only line of evidence to be considered in arriving at the present value of properties devoted to public use. Past costs always will be considered, both of themselves, and also in determining the cost of reproduction new of the comparative plant, the conceptional construction of which is necessary in determining reproduction costs as of to-day. It must be admitted that the reproduction new method involves largely questions of opinion which are evolved from the consideration of all relevant facts. The past cost method, however, is purely a question of fact. That is the fundamental difference between the two methods.

The writer does not wish to be misunderstood as advocating the promiscuous intermingling of past and present costs in arriving at the fair present value of a property. This is, perhaps, the most frequent error that is to be found in utility appraisements. He simply wishes to emphasize the value of past costs, especially of the component parts of the properties, as the best evidence which may be obtained to determine the present costs of reproduction new. From past costs of units, where available, one may very well reason to present costs. Without such past costs, all evidence of present cost is purely comparative, and, consequently, is a matter largely of opinion. Therefore, it is open to more or less serious objection or criticism, in so far as it approaches the realm of conjecture.

In conclusion, the writer believes that engineers must accept and adopt the position, so well argued by Mr. Alvord, that cost of reproduction new, as of to-day, is a principal line of evidence to be investigated in all appraisals of properties devoted to public use. He wishes to emphasize the fact, however, that this is not the only line of evidence to be considered, and that past costs, especially of recently constructed properties, afford the best evidence as to the cost new of many items which enter into the estimated cost of reproduction, as of to-day. Past costs of entire properties generally are not available, and, where they are available, may be very misleading if used to determine "the fair present value of the property used and useful to the public."

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PAPERS AND DISCUSSIONS

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THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA.

Discussion.*

BY MESSRS. JAMES H. HARLOW, R. D. COOMBS, HUGH L. COOPER, HEINRICH HOMBERGER, H. BIRCHARD TAYLOR, SPENCER MILLER, AND W. E. MITCHELL.

Mr. Harlow.

JAMES H. HARLOW,† M. AM. SOC. C. E. (by letter).—Like most writers of technical papers, the authors fail to describe the plant constructed so as to permit the reader to answer the question "Will it pay?" This is the question uppermost in the minds of those to whom we go for funds with which to carry out our ideas. It involves two other questions, *viz.*, the cost of the plant under consideration, and the character and extent of the market for the power to be developed.

The writer finds one stock objection to the construction of a hydro-electric plant, and that is, "a steam plant can be built for much less, and the best turbo-generators can be operated on 20 lb. of steam per kw-hr." Several times when this statement has been investigated, it has been found that 20 lb. of steam did not include the auxiliaries, and instead of 20 lb., it was nearer 30 lb.

The writer is interested in several hydro-electric plants, and, when the stream will give power for 9 months per year and for the other 3 months the plant is operated by steam, the resulting power is produced more cheaply than by steam alone.

A year has passed since the construction of the Lock 12 Development, and some interesting facts must have accumulated in the opera-

* This discussion (of the paper by E. L. Sayers and A. C. Polk, Members, Am. Soc. C. E., published in September, 1914, *Proceedings*, and presented at the meeting of November 4th, 1914), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

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Mr. Harlow.

tion of the plant. The authors will supplement the paper, perhaps, by answering the question, "Has it paid?"

What was the cost (ready to operate) of the hydro-electric plant per kilowatt on the station switch-board?

What was the cost of the steam auxiliary plant per kilowatt on the station switch-board?

(The cost per kilowatt should be based on the actual annual output of each plant.)

What was the actual output of each on the switch-board?

The greater the detail of the costs, the more value it will afford to designing and operating engineers.

Another point of interest is the size of the units. The authors say, "The adoption of this size of installation was dependent on many factors and a number of assumptions, * * *." May we hear further on this point later?

The writer has knowledge of a plant designed to adopt units of 37 500 kw. Are we not trying to follow a fetich in these large units?

From some of the writer's investigations, there is a point in the cost of generators where the cost per unit of output rises. Is not the cost of the combined hydraulic and electric machinery a rather small part of the total development cost, and should it be followed as far as we seem inclined?

At the plant in question, can more than one unit be operated at the extreme low-water flow, and if two must be run to get the use of all the water flowing, is not the efficiency being reduced much below that expected?

The highest efficiency is in demand at times of low water; when there is plenty of water, efficiency is not so necessary. The writer has demanded that the highest efficiency shall be at about from 75 to 80% of gate-opening. Turbines cannot be well regulated when operating at near full gate.

How are the authors going to determine whether or not the "guaranteed efficiency" is obtained? They certainly do not depend on the Holyoke tests of a (comparatively) small turbine and then the guess of the builder that the full-sized turbine will "fill the bill."

The best hydraulic tests are probably not within several per cent. of the truth. Do we know enough about the "chemical test" to be sure we know the exact efficiency?

Horizontal vs. Vertical Units.—Unless the tail-water rise is in excess of the suction effect, why was the vertical unit adopted instead of the horizontal?

Some years ago we required our steam turbo-generators to swing on a vertical axis, but now we are going to the horizontal.

The hydro-generator is now going toward the vertical; shall we, after a while, swing back to the horizontal?

Mr. Harlow.

The McCall's Ferry plant has been referred to as using the vertical type, but here the tail-water rise may be as much as 35 ft., and it must be remembered that the third thrust-bearing is being tried, and successfully thus far; but it took the third trial to get the bearing. A longer trial may change our views.

Papers like this are valuable, but more attention should be given to "will it pay" or "does it pay," and to do this we must have details of the costs as well as of the construction.

Mr. Coombs.

R. D. COOMBS,* M. AM. SOC. C. E.—As it is the avowed purpose of the authors to bring out discussion on the latest ideas and methods relating to hydro-electric power plants, including those pertaining to transmission line construction, the speaker will confine himself to a discussion of, and perhaps some additions to, the methods described in the paper.

Unfortunately, the authors have followed the usual practice and dismissed the subject of line construction in a very sketchy way. The word "unfortunate" is used advisedly, as the subject merits more consideration.

Apart from the real need of more literature on this subject, in order that future work may have the benefit of past experience, it would seem that the sole link connecting the expensive power-generating station with the consumer should be considered of relatively greater importance.

In addition to the two foregoing considerations, the speaker believes that the comparative importance of the line, from an economic standpoint, has usually been partly disregarded. As the entire installation is a commercial enterprise, the income of which is derived from the sale of the current carried by the transmission lines to the purchasers, the lines themselves may be a large factor in the possible quantity of unproductive, expense-bearing time. In other words, the connecting link may cause stoppages in the sale of current, and, in extreme cases, render the entire installation unproductive for various periods of time. Therefore, the maximum possible security from interruptions to service, obtainable with a reasonable expenditure, is very desirable in any line installation.

The transmission lines—and in this discussion no special distinction will be made between so-called transmission and distribution lines—may be of two general classes: aerial open wires and underground insulated wires or duct lines. The latter are approximately three times as expensive to build as the former, and as they have but few applications in hydro-electric development, in the United States, may be omitted from consideration herein.

In reviewing the data in the paper, it will be noted that in general the design appears to follow the latest methods of wide-base tower

* New York City.

Mr. Coombs.

construction. These are—to give them in outline—long spans, stranded cable, overhead ground wires, vertical spacing, and staggered middle cross-arms. Unfortunately, information is lacking as to the details of construction, and the reasons for various decisions as to such details.

Referring to the lengths given for the two kinds of cross-arms, it would seem that there must be an error in the stated length of the middle arm, because, as given, it only offsets the wires 9 in. This small offset, which is introduced in such lines in order to minimize the probability of wires sagging, falling, or blowing into contact with each other, is inadequate for the purpose.

It is stated that the Type A towers, which are 68 ft. high, were used in rough country with standard spans of 700 ft., and that Type C towers, 65 ft. high, were used in flat country, with standard spans of 600 ft. Unless the towers could be located in an almost miraculously favorable manner, the speaker believes that this relation would not obtain, because rough country ordinarily compels the use of spans shorter than standard, unless the towers are made higher. This is due to the constant need of decreasing the sag in order to maintain the required clearance of the wires above the ground.

Although not thus stated, the inference may be made that the design, as well as the clearance, was based on a loading of $\frac{1}{4}$ in. of ice and 8 lb. of wind pressure. The speaker believes that a loading of $\frac{1}{2}$ in. of ice and a wind pressure of less than 8 lb. is better general practice. Some reduction from the latter loading may be justified in certain cases, particularly in construction with very long spans, but the speaker believes that in actual practice it will be the wind pressure, rather than the ice, which will be reduced. However, in the case in question, it may perhaps be argued that a large accumulation of sleet need not be expected on 110 000-volt wires while alive, and that the period during which the wires will be dead would reduce the probability of the maximum load to a negligible quantity. It would be of interest if further explanation were given in regard to the relative strength of the towers in the standard spans, and in the maximum spans mentioned. Ordinarily, such maximum spans are of rare occurrence, and they are frequently guyed in order to withstand the heavier loads.

Although no particular criticism may justly be made of the main transmission line, as it fairly represents one of the two best types of construction, there may be some difference of opinion as to its relative advantage as compared with two one-circuit pole or tower lines. The so-called distribution line, of 22 000 volts, on wood poles, however, is not of as excellent construction, and, if its location is permanent, a steel pole line would perhaps have been justified instead.

As steel pole lines, such as those shown by Fig. 54, with spans

FIG. 54.—TWO-CIRCUIT STEEL POLE LINE.

of medium length, are growing in favor for 22 000-volt transmission lines, they are worthy of consideration for a 22 000-volt distribution line. The size of the conductors in the 22 000-volt line is not stated, and a photograph is an uncertain basis for measurement, but the pole spacing of 175 ft. and the vertical and horizontal spacing of the conductors do not appear to be consistent. Mr. Coombs.

HUGH L. COOPER, M. AM. SOC. C. E.—The cross-section of the wheel setting, Fig. 11, shows that the thrust-bearing is carried on top of the generator frame. It would be interesting to know the experience with the thrust-bearings at this plant, particularly with reference to their ability to carry the load without interruption. The speaker has been informed recently that, in operating under conditions approximating full load, these bearings have given trouble. Mr. Cooper.

These Kingsbury bearings have been used with much success in the plant of the Mississippi River Power Company, at Keokuk, and the speaker believes that they are, undoubtedly, the coming bearings for vertical shafts with heavy loads, provided they are supported properly. Exact data regarding any trouble with this bearing at the Coosa plant would be valuable. On more than one occasion the proper location of the thrust-bearing on the shaft has been under discussion. Some have been of the opinion that such bearings can be supported on long metal spans and run successfully; others hold that they should never be placed in that way, but should be supported on a non-elastic foundation—the firmer the better. Specific and correct data on this subject will be of much value in designs for future work of this kind.

In reference to the pit liner, Fig. 13, will the authors state whether or not the castings were machined after they were set in concrete? The speaker has always found that the shrinkage of the concrete threw the casting out of line, no matter how well it was milled in the shop. On the work at Keokuk the pit liners were set with the greatest possible care, using specially designed leveling jacks, and notwithstanding the great weight and stiffness of these liners, there was a considerable change in the alignment after the concrete had set. This could not be neglected, and therefore the main bearing surfaces were milled *in situ*.

HEINRICH HOMBERGER,* M. AM. SOC. C. E. (by letter).—This very interesting paper is without doubt one of the most valuable additions to the literature on large, low-head, hydro-electric developments of the most recent type. Mr. Homberger.

In reference to the turbines, the authors state that one of their features is that they are "self-contained." With first-class design, vertical-shaft turbines of this particular type, of such huge dimensions, could hardly be otherwise; each, of necessity, must be a homogeneous

* Mill Valley, Cal.

Mr. Homberger.

structure, or it would be impossible to make a satisfactory job of the installation in the plant. It is hardly possible to believe that any modern manufacturer would contemplate building such a turbine without setting it up completely in the shop before shipment to the plant, and, in order to clear up what the writers had in their minds, it would be interesting to note what character of "very expensive machine work is commonly done in the field" with turbines of the size and type described.

The turbine runner is fastened to the shaft with a taper fit. This is very expensive work if thoroughly well executed. A taper fit has the advantage that the joint may be readily broken, which is of importance where the hub is to be removed from the shaft frequently. This is hardly expected to be the case with a turbine runner. If a parallel press fit is selected instead, it has the advantage that the runner can be driven home to a fixed point and its location is absolutely determined. Of course, there is no leeway in the location of the runner on the shaft, and to make the taper fit a success, it is generally necessary to press the runner on and off the shaft many times, with intermediate scrapings, until the hub is exactly in its right position on the shaft and at the same time comes to a perfect bearing thereon. It seems that the taper fit is rather a design borrowed from marine practice and used for fastening a propeller to the tail-shaft. Propellers are likely to have to be replaced much more often than turbine runners, and the exact location in the axial direction of the shaft is of no importance. It is sufficient to scrape the propeller hub to a fit on the shaft and then tighten the nut.

With reference to the efficiencies which have been guaranteed: Supposing that the generator was expected to operate on a 75% power factor, this would allow about 25% over-load capacity in the turbine. It is common practice to select the point of highest efficiency at about three-quarters of the capacity of the turbine, which would be somewhat below the rated capacity of the generator, because, at a plant with several units, the machines are expected to operate normally at about the rated capacity of the generator. With the Alabama turbines, the highest point of efficiency appears to have been very near the full capacity of the turbine (the exact point is not given), and it is very rarely expected that the turbine will operate under such a heavy load; in reality, it would probably be operated mostly near 14 000 h.p. output where the efficiency is 4% lower than the guaranteed maximum.

The question of the value of Holyoke tests compared with field tests after installation has been thrashed out frequently, and the writer was pleased to see how much stress the authors have laid on the necessity of checking up the Holyoke results by actual field tests. Although Holyoke tests are very interesting to the designer of turbines,

Mr. Homberger.

they give an insight only into the characteristics of the turbine runner and its behavior under varying conditions; all the other accessories which make up the complete turbine, and the general mechanical and hydraulic conditions of the setting, which greatly influence the efficiency, cannot receive any consideration at the testing flume.

The question whether preference should be given to vertical or horizontal units has been argued, and the selection of the former is explained by stating that horizontal units would promise not as high an efficiency on account of the necessity of sharp bends in the draft-chest, causing loss of head, and that there were other losses from the confluence of streams in the draft-tubes of multi-runner horizontal units.

To the writer, it does not seem necessary to make the curvature of the draft-chest of a single-discharge horizontal turbine of any smaller radius than the draft-chest of a vertical unit. Referring to Fig. 7, which shows the draft-elbow as constructed in concrete masonry, exactly the same curvature could have been chosen for a single-runner, single-discharge turbine with horizontal shaft. As far as space is concerned, if the units were arranged athwart the power-house, the greater part of the draft-chest could be embodied in the power-house substructure, and the length of the power-house would not have to be increased. If such a unit had been selected for the Lock 12 Development as described, considering the very large capacity of the units, it would probably have developed into a two-bearing independent turbine coupled to a two-bearing engine-type generator. It might have been advantageous to combine generator and turbine into a three-bearing unit with one continuous shaft, requiring a narrower power-house. In either case, although horizontal machines would have required a wider building, the foundations would have been very much simpler than they necessarily had to be for vertical units; and a comparison of costs of the two types of power-house building would be interesting to the profession. Doubtless such a comparison was made by the engineers of the power company before units of the vertical type were adopted.

One of the prime advantages of the horizontal-shaft unit is its greater accessibility and the greater ease with which the turbine may be dismantled. In the natural course of events, turbine runners of any type of unit are bound to wear, and must be replaced sooner or later. With a horizontal shaft and a scroll casing built up of a number of parts, the removal and replacement of a runner is much easier and much less costly, and takes a great deal less time than with a vertical-type unit where the entire stationary superstructure of the generator, the exciter, the thrust-bearing, the steady-bearing, the gate rigging, and the entire rotating part of the unit have to be dismantled before the turbine runner may be removed from the shaft and replaced by a new one.

Mr. Homberger.

Highest efficiencies and economic construction may well be attained with volute or scroll casings of large horizontal shaft units, as recent practice both in the United States and in Europe has proved.

It does not seem impossible to the writer to design a horizontal, single-runner, single-discharge unit for the conditions as they are in the Lock 12 Development and to obtain satisfactory structural features throughout.

With reference to "loss from the confluence of streams in the draft-tubes in multitudinal units", this does not seem to apply in the present case. If a single-runner, double-discharge unit had been chosen, these losses could have been avoided by arranging for two draft-tubes, and there would have been no confluence of streams. The question of installing units of the double-discharge and single-runner, or double-discharge double-runner type, had probably been examined to a considerable extent when the hydraulic equipment of the plant was under discussion. There could also have been selected a unit with two scroll casings, one for each runner, which would have promised a very high efficiency. This type of unit has become well known to the profession since the power-plant of the Ontario Power Company at Niagara Falls was started, about a decade ago. The prime advantage of this type of machinery would have been the possibility of selecting a turning speed about 50% higher than the one resorted to in the Lock 12 plant, resulting in the advantage of a less costly generator, and lighter parts to be handled, and indirectly in a saving in cost of installation, overhauling, and replacement. It would be interesting to learn what comparative investigations had taught in connection with the Lock 12 Development, prior to the adoption of the vertical units.

Another point in favor of the horizontal unit is the fact that the gate-operating mechanism, governor connections, etc., permit a much more convenient arrangement.

The question as to the highest efficiency which may be obtained with a scroll casing does not seem to be entirely settled yet; as a matter of fact, turbine runners tested at Holyoke in an open flume have sometimes shown higher and sometimes lower efficiency than they did finally in the plant when installed with the scroll casing for which the runners were originally designed.

Mr. Taylor.

H. BIRCHARD TAYLOR,* ESQ. (by letter).—This very complete paper is a valuable contribution to the literature on modern hydro-electric plants. The progress made in the last few years in this field, particularly in respect to the turbines, has been very marked, but the literature on this subject has not kept pace with it; so that it is a real pleasure to read this paper. It not only describes one of the most modern and prominent developments in the United States, but

* Hydraulic Engr., I. P. Morris Co., Philadelphia, Pa.

Mr. Taylor.

also gives a considerable amount of valuable technical detail, which should prove of interest to engineers.

The writer is particularly interested in this development, as the turbines were designed under his direction. The following comments will be confined to that subject.

The contract for the four 17 500-h.p. turbines was awarded in October, 1912, a short time after the single-runner, vertical-shaft, low-head turbine had come into use, and during the early stages of its development. At that time there had never been constructed a vertical-shaft, single-runner unit of so high a power to operate under a head as low as 68 ft., nor had there been used concrete volute turbine casings under a head as high as 68 ft. In two respects, therefore, the Alabama turbines represent a pioneer development.

Previous to the award of the Alabama contract, there had been a few plants completed or under construction in which the vertical-shaft, single-runner unit had been adopted, and in which the volute casings and draft-tubes were moulded in the concrete. Among these may be mentioned the two plants of the Appalachian Power Company, in Virginia; the plant of the Mississippi River Power Company, at Keokuk, Iowa; and the plant of the Georgia-Carolina Power Company, in Georgia. The turbines in these plants, however, were designed for lower heads and for much lower unit capacities. At this writing, the Alabama units are the most powerful vertical turbines of this type in existence, with one exception; namely, the 20 000-h.p. units of the Laurentide Company, Limited (head 76 ft.).

Table 7 gives a comparison of the powers, heads, and speeds in the plants just mentioned.

TABLE 7.—Comparison of Several Hydro-Electric Plants.

	Head, in feet.	Speed, in revolutions per minute.	Capacity, in horse-power.
Appalachian Power Company, Plant No. 2	49	116	6 000
Appalachian Power Company, Plant No. 4	34	97	3 500
Mississippi River Power Company	32	57.7	10 000
Georgia-Carolina Power Company	27	75	3 125
Alabama Power Company	68	100	17 500

The Appalachian turbines were the first of this type to be put into operation, and the results were so satisfactory that the principal features of their design were adopted for the Alabama turbines, and have been used in a very large majority of recent installations.

The Appalachian turbines were tested after the award of the Alabama contract. The average of the maximum efficiencies secured in two units was found to be 93.7%, which, at that time, seemed almost

Mr. Taylor.

incredible, as this result was about 4% greater than the maximum efficiency secured in the model runner at Holyoke. This increase in efficiency over the model runner, as obtained in the vertical, single-runner setting, however, has been recently confirmed by the tests conducted in the plant of the Mississippi River Power Company, at Keokuk. The efficiency secured in the Appalachian turbines, along with the mechanical simplicity of the vertical-shaft, single-runner unit, has established that type for all important low and medium head installations.

The test of the Appalachian turbines should be of peculiar interest to the engineers of the Alabama Power Company, for the reason that the designs of the runners in the Alabama and Appalachian units were based on the same Holyoke model. As the settings of the turbines in both plants are almost identical, the efficiency of the Alabama units may be assumed to be at least as high as that secured in the Appalachian units, and probably slightly higher, in view of their greater size and higher power.

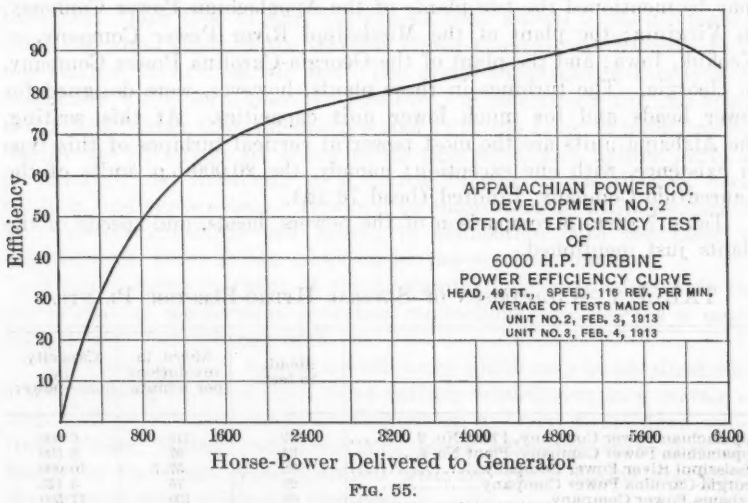

FIG. 55.

Fig. 55 shows the official efficiency curve of the Appalachian turbines. If we revise the horse-power scale, and substitute 17 500 h.p. for 6 000 h.p., the curve may be assumed to apply to the Alabama turbines. When the turbines are tested, it will be very interesting to learn just how closely the efficiencies obtained approach this curve.

During the past 2 years there have been constructed a great number of vertical-shaft, single-runner units for prominent developments —the I. P. Morris Company alone having constructed turbines of this type aggregating approximately 500 000 h.p.—but there have not

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been any new features of importance developed since the design of the Alabama turbines. The lignum-vitæ guide-bearing; the speed ring; the direct-connected operating engines with trunk pistons; the location of the thrust-bearing above the generator; the use of the Kingsbury thrust-bearing; and the pneumatic brakes arranged to act against the lower face of the rotor, etc., have been pretty well accepted as standard details in subsequent work.

The Alabama Power Company was one of the first to adopt the speed ring in connection with concrete volute casings. The design of the speed ring for these turbines was unusual, because of the high head involved. Aside from making the speed ring vanes of sufficient strength, when acting as columns, to carry the weight of the concrete floor above the casing, the weight of the pit liner, generator, and thrust loads down to the concrete below the casing, it was necessary to make them safe when subjected to tension. The rise in pressure in the volute casing during sudden closure of the turbine gates, when added to the already high normal pressure, would tend to lift the concrete floor above the casing, which must be resisted by tension in the speed ring vanes. The speed ring vanes, therefore, were made amply safe to withstand the greatest possible load in either direction which might be imposed.

The use of the speed ring, in connection with the Appalachian turbines, was the subject of considerable adverse criticism. The point was often raised as to whether the speed ring vanes, which are fixed, would not tend to peak the efficiency curve and reduce part load efficiencies. In other words, it was pointed out that these fixed vanes did not conform to the guide-vanes at all gate openings, but were suited to one gate opening only. The Appalachian curve, as shown in Fig. 55, shows no tendency to peak, and the part gate efficiencies cannot be said to be low. It should be remembered that at low gate openings, although the speed ring vanes do not come opposite the guide-vanes, as they do at or near the point of maximum efficiency, the velocity of the water is low, and a comparatively sudden change in the direction of flow when entering the guide-vanes is of no great moment.

It was also argued that the usual experimental outfit in which the Holyoke models are tested is not equipped with speed rings, and, as very high efficiencies have been secured at Holyoke, there is no reason for the use of speed rings in large turbines. Although it is true that very high efficiencies have been secured at Holyoke, the foregoing conclusions are not justified. At Holyoke we have an open flume, in which the velocities are comparatively low, though, in the case of large units with spiral cases, the usual velocities are quite high and represent very much higher percentages of the spouting velocity than those which exist in the Holyoke flume.

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The principal advantages secured in the use of the speed ring are:

1st.—A speed ring insures a uniform direction of flow to all the guide-vanes. Although, as mentioned previously, this is not of importance at low gates, it is at the gate openings at and above those corresponding to maximum efficiency, where the velocities are high.

2d.—The projected area normal to the direction of flow of the speed ring vanes is materially less than that of tie-bolts. The lines of flow, therefore, are interrupted to a less extent by the speed ring vanes than by tie-bolts of circular section. The writer believes that when high velocities are encountered, the projected area normal to the direction of flow is about as important a consideration, as far as losses are concerned, as the actual shape of the tie-piece.

3d.—From an economical standpoint, speed rings are desirable, because without sacrifice in efficiency, they permit higher velocities in the turbine casing, therefore, closer unit spacing and, consequently, a reduction in power-house dimensions.

4th.—The mechanical advantages derived from the use of the speed ring are sufficient for its adoption, aside from any hydraulic advantages. In erection, it is desirable to eliminate machine work and unnecessary fitting in the field. In the construction of the turbine in the shop, the speed ring is put on a boring mill, and, at one setting, is turned to suit the runner band, the lower distributor plate, the head cover, and the lower section of the pit liner, so that, when it is leveled up in the field, the location of all these connecting parts is definitely established.

The advantages which the speed ring offers in securing quick and accurate alignment of the entire turbine during erection, over any design involving separate foundation rings with independent tie-bolts, requiring the separate alignment of each ring, are obvious.

A comparison of the erection costs of two types of turbines, installed simultaneously by the same company, one with speed rings and one with tie-bolts, showed a very marked advantage in favor of the former type. As the units were otherwise similar, the saving in cost as well as the greater speed in erection was attributed to the advantages secured by the use of speed rings.

During the past 2 years the merits of the speed ring have been very generally recognized, and it has been specified by the engineers of a great number of prominent developments. It is interesting to note that the manufacturers who at first argued strongly against its use have recently provided it in connection with some of their contracts.

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The authors' remarks, regarding turbine efficiency guaranties, as stated on page 2221*; are not altogether clear. They state:

"And the manufacturers have guaranteed that the units as installed will have a maximum efficiency of 87% when delivering to the shaft from 15 000 to 17 000 h.p."

This statement should read as follows:

"The manufacturers have guaranteed that the turbines will have a maximum efficiency of at least 87% when delivering a power between 15 000 and 17 000 h.p."

In other words, the turbines are not guaranteed to develop 87% efficiency for all powers between 15 000 and 17 000 h.p., but are guaranteed to develop a peak efficiency of 87%, such peak to occur between the limits of 15 000 and 17 000 h.p.

In making the preliminary studies of the Alabama units, the question of speed was given very careful consideration. The principal features studied in determining the speed were as follows:

1st.—The allowable elevation of the center line of the runner above low tail-water.

2d.—The corrosion of the vanes of high specific speed runners when operated under high heads.

3d.—Part-gate efficiencies.

4th.—Powers developed under heads below normal.

These questions will be considered in order.

1st.—Elevation of Runner Above Tail-Water.—With the development of high specific speed runners, the question of draft-tube design and the allowable elevation of the runner above tail-water has been given very careful attention. Until comparatively recently, it has been thought that the draft-tube was merely a device by which the turbine could be placed a certain distance above tail-water without losing the corresponding head, and it was generally thought that this distance was only limited by the height of the barometric column.

The draft-tube to-day, however, is looked on as one of the most important elements entering into the turbine, especially where high specific speed runners are involved. It is a function of the draft-tube, not only to enable the runner to be placed above tail-water level, but to regain the energy existing in the velocity of discharge from the runner vanes. The off-flow velocity head from high specific speed turbine runners may be as high as 50% of the total head acting on the turbine. The regaining of this velocity head by the draft-tube takes place in the form of a reduction in pressure or an increase in vacuum in the draft-tube, the maximum vacuum occurring immediately below the runner, where the discharge velocity is greatest.

* *Proceedings*, Am. Soc. C. E., September, 1914.

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With a draft-tube of 100% efficiency, the vacuum at the runner is equal to the static draft head or vacuum corresponding to the vertical distance to tail-water level, plus the head corresponding to the velocity of discharge from the runner. The sum of the static draft head plus the velocity draft head must not be greater, theoretically, than the barometric column (34 ft. at sea level). Actually, this sum should not approach this limit by less than 3 or 4 ft. The sum of the static head and velocity head in the draft-tube is, of course, reduced by the losses in the draft-tube and also by the head corresponding to the velocity at the discharge section of the draft-tube, but in modern plants these two elements may be disregarded, as they are kept within negligible values.

The margin between the allowable draft head and the theoretical maximum draft head, is provided principally for regulating purposes. Due to the inertia of the water column in the draft-tube, the sudden closing of the turbine gates increases momentarily the vacuum at the runner, and if this vacuum before closure is already up to the limit, the column of water may leave the runner and then return in a very strong upward surge. In some cases, this upward surge in the draft-tube has resulted in lifting the rotating element of the unit clear from the thrust-bearing.

If flood conditions make it necessary to place a turbine runner, say, 20 ft. above low tail-water level, the velocity draft head at the discharge of the runner at full load should not exceed, say, 10 or 11 ft. With an increase of specific speed, the velocity head through the band, or throat of the runner, must increase, because a runner, to develop a given power, decreases in diameter with an increase in specific speed. Therefore, a specific speed must be selected which will not give a greater velocity head from the runner than that equal to the difference between the 3 or 4 ft. less than the barometric column and the static draft head.

The static draft head of the units at Lock 12; that is, the vertical distance from the center line of the intake to the runner to low tail-water, is 20 ft. With an assumed efficiency, at 17 500 h.p. and 68 ft. head, of 90%, the discharge velocity head from the runner amounts to 10.4 ft. Under the maximum head of 74 ft., at the same gate opening, this velocity head would be increased to approximately 11.3 ft. The resulting maximum draft head possible is, therefore, 31.3 ft.

2d.—The Question of Corrosion as Affected by Specific Speed.—Unfortunately, there is no established law for determining the maximum specific speed allowable for any particular head, aside from that affecting the draft head, which has just been discussed.

As the specific speed of a runner increases, the width across the distributor or intake increases, and the diameter decreases. In addition to the velocity of discharge from the runner increasing with an

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increase of specific speed, there are conditions set up, primarily at part gates, which make runners of this type very much subject to corrosion, caused to a great extent by the radical change in the direction of the flow lines for changes in gate opening.

Aside from the question of draft head, it may be said that the safest method of determining the allowable specific speed for a given head is by consulting a curve drawn between specific speed and head, on which are plotted points corresponding to various installations, which have been in operation for a considerable time and are known to be successful in withstanding corrosion. This curve may be termed the "curve of experience".

In considering turbines for a new development, therefore, the revolutions per minute adopted should not call for a specific speed which would fall to any great extent above a curve thus drawn. It is advisable, of course, to reach out gradually from this curve, or, in other words, to extend our experience, but no great step should be taken at any one time.

At the time of the selection of the speed for the Alabama turbines, it was known that turbines in several installations had successfully withstood corrosion under conditions approximating those which would exist in the Alabama units when operating at a speed of 100 rev. per min.

The next two questions will be considered together:

3d.—The Question of Part Gate Efficiencies as Affected by the Revolutions per Minute of the Unit; and 4th.—Power at Low Heads.—One of the inherent characteristics of the performance of turbine runners is a decrease in the part load efficiencies with an increase in specific speed. There is, of course, a limit to this rule. In other words, by decreasing indefinitely the value of the specific speed and, consequently, the revolutions per minute of the unit, the part load efficiencies apparently cannot be raised above certain maximum values. As far as part load efficiencies are concerned, there seems to be an ideal specific speed by the adoption of which it is possible to obtain part gate efficiencies which cannot be improved on by further decrease in specific speed. The experience of the writer would indicate that this specific speed resulting in an ideal shape of efficiency curve, as far as part gate efficiencies are concerned, lies in the neighborhood of 120 (metric).

Fig. 56 will convey an idea of the manner in which these part gate efficiencies increase with decreased specific speed. The values of the specific speeds shown in this figure are in the neighborhood of that of the Alabama turbines (301.5). It will be noted that Curve *O* is of the highest specific speed, namely 370; that it develops a maximum efficiency of a trifle more than 90%; that the part gate efficiencies are lower than those of the remaining curves, and that

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the point of maximum efficiency occurs nearer full load than is the case with the remaining curves. Curve *M*, with a specific speed of 290, has a much higher part gate efficiency and also a slightly higher maximum efficiency, the point of this maximum efficiency occurring at a lower percentage of full load than on the remaining curves. Curves *A* and *R* represent specific speeds of 301 and 339, respectively. The shapes of these curves are consistent with those of *M* and *O*.

In applying the question of selection of speed directly to the Alabama turbines, it may be of interest to reproduce the performance curves which were discussed with the Alabama Power Company. These curves are shown in Fig. 57. It will be noticed that there are four efficiency-power curves, *M*, *E*, *A*, and *O*. These curves are stepped up from runners designated by these letters. The runners on which these curves are based, when stepped up to give the power indicated, would operate at speeds of 94.8, 100, 100, and 120 rev. per min. in the case of types *M*, *E*, *A*, and *O* runners, respectively.

It was decided that though the selection of Type *M* runner would result in extremely high part gate efficiencies, the fact that this runner would not give high power at heads below normal and that the revolutions per minute resulting would necessitate a comparatively expensive type of generator, caused its elimination.

The speed possible with a runner similar to Type *O*, namely 120 rev. per min., was very desirable, but the part gate efficiencies secured were so low, when compared with the others, as to make its adoption unwarranted. Also, the selection of a runner having so high a specific speed would possibly have rendered the runners susceptible to corrosion, as discussed previously (being too great an advance beyond the "curve of experience"); also, it would have been necessary to place this runner at an elevation 5 ft. below that fixed for the runner actually used, and this was found to be objectionable.

The comparative virtues of Runners *A* and *E* were next considered. As both these runners were capable of delivering the same power at the same revolutions per minute, the question of speed, of course, did not figure. The advantage of higher part gate efficiencies of Runner *E* was, it was thought, partly offset by the fact that this runner developed under the lower heads less power than Runner *A*. Further, the dimensions of Runner *E*, when stepped up to the Alabama conditions, resulted in a considerably higher width of distributor, or height of guide-vane. Also, Runner *E*, when enlarged to suit the Alabama conditions, would be materially heavier and, therefore, more costly than Type *A*. The fact that the latter runner made possible a very low width of distributor and a runner of very inexpensive design, and also the fact that it developed under the low head conditions comparatively higher power, resulted in its selection.

One or two thoughts have suggested themselves to the writer in regard to the governors.

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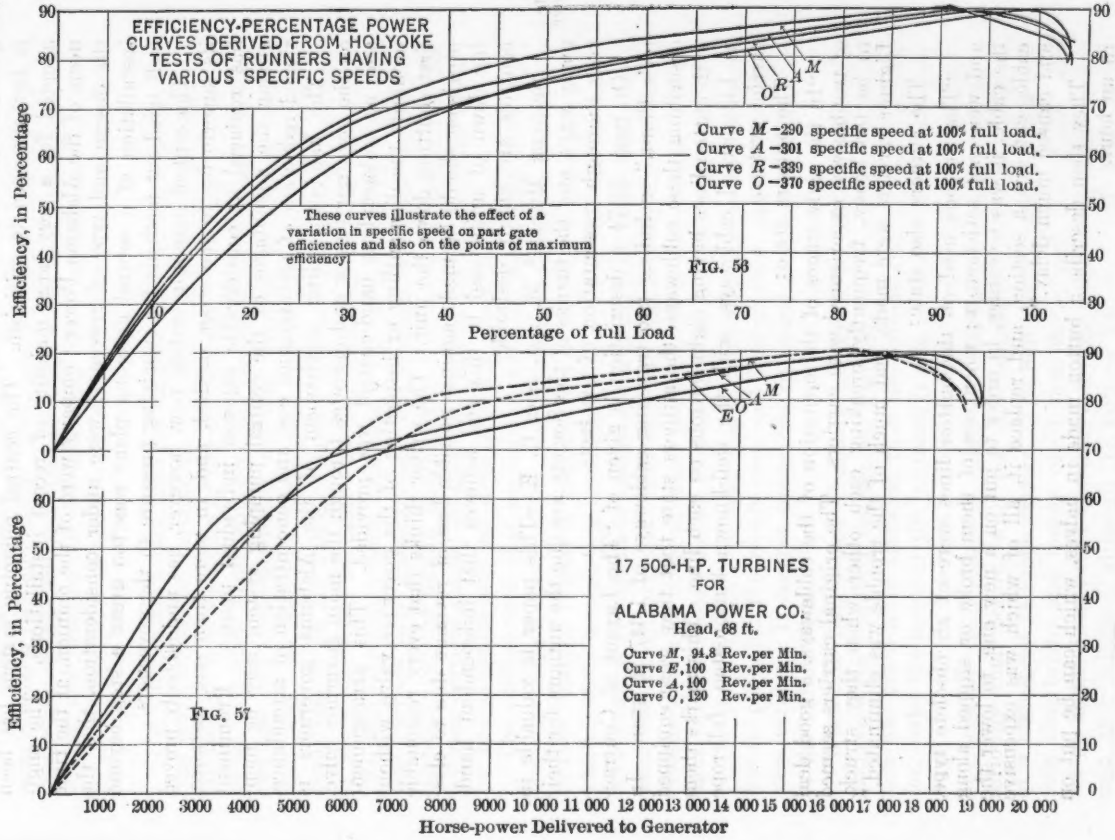
EFFICIENCY-PERCENTAGE POWER CURVES DERIVED FROM HOLYOKE TESTS OF RUNNERS HAVING VARIOUS SPECIFIC SPEEDS
Curve M —290 specific speed at 100% full load.
Curve A —301 specific speed at 100% full load.
Curve R —339 specific speed at 100% full load.
Curve O —370 specific speed at 100% full load.
These curves illustrate the effect of a variation in specific speed on part gate efficiencies and also on the points of maximum efficiency.
Efficiency, in Percentage
Percentage of full Load
O R A M
FIG. 56
17 500-H.P. TURBINES FOR ALABAMA POWER CO.
Head, 68 ft.
Curve M, 94.8 Rev.per Min.
Curve E, 100 Rev.per Min.
Curve A, 100 Rev.per Min.
Curve O, 120 Rev.per Min.
E O A M
Efficiency, in Percentage
Horse-power Delivered to Generator
FIG. 57

Mr. Taylor. The adoption of individual pumps for supplying pressure to the governors in a plant containing as many as four units (six eventually), is perhaps open to criticism. The central pumping system has been adopted for a very large majority of recent installations. The engineers of the Alabama Power Company were of the opinion, at the time the design and type of governors were under consideration, that the installation of a central pumping plant was too great a step beyond well tried-out methods for supplying pressure to the governors.

The central pumping system now, however, has undoubtedly proved its superiority to the other method, and, in more than one instance, has replaced at considerable expense individual pumps. Prominent among the advantages of the central pumping system may be mentioned reduction of maintenance cost and concentration of attendance.

The hand-controlling mechanism of the Alabama governors is on the governor proper and operates through the main governor valve, and no independent hand-control was provided. This arrangement prohibits the dismantling or inspection of the governor valve without first shutting down the unit. The writer thinks that every reasonable provision should be made against possible loss of revenue due to closing down of units, and to this end believes that independent hand-controls are highly desirable.

Mr. Miller. SPENCER MILLER,* M. AM. SOC. C. E.—This paper is valuable in many ways, and the thanks of the Society are due the authors for their very thorough presentation of the facts.

On page 2247† a description is given of "The Layout of Contractor's Plant," including two traveling cableways of 1 647 ft. span. In describing these cableways, the authors state that they were equipped with button lines having eight buttons on each line. This marks them as Lidgerwood cableways, with the well-known button-stop, fall-rope carrier system.

The authors state:

"In the early stages of the operation of the cableways, a good deal of trouble was experienced with carriers. The original carriers seemed to be too heavy, frequently breaking each other when they struck. Lighter carriers were made, and much of the trouble was eliminated."

The authors also state:

"The buttons used on the button lines were of an obsolete type, and were not satisfactory; when one of them broke or slipped along the cable, it was necessary, in order to put on a new one, to lower the cable, cut out a section, and replace it, all of which was expensive and caused much delay."

They then describe a button made in halves, which can be put on in an hour.

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* New York City.

† *Proceedings*, Am. Soc. C. E., September, 1914.

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Again, the authors state:

"The cableways handled practically 60% of the concrete, and were run both day and night. They also handled practically all the excavation in the early stages of the work. They were used for moving and erecting all derricks, etc., handling form lumber, iron, and materials of a like character, and were found to be exceedingly useful for the latter purpose."

The authors have said that the buttons used for spacing the fall-rope carriers were of the obsolete type, and it is probably true that the fall-rope carriers were also of the obsolete type. The results, no doubt, would have been wholly different had the modern high-speed cableway carriage and shock-absorbing fall-rope carriers, as well as buttons, been used. As indicated in the paper, the efficiency of a cableway depends largely on the fall-rope carrier system. The system mentioned by the authors was probably identical with that used on the Chicago Drainage Canal on twenty cableways. The repair costs for these fall-rope carriers were considerable. The constant attention required in strengthening the button fastenings and the slots in the head of the fall-rope carriers showed an inherent fault in the design. The remedy for this difficulty was not discovered for many years.

The authors state that the carriers were too heavy and broke when they struck each other. This they say was remedied by making the carriers lighter. Impact is in proportion to weight. A fall-rope carrier which is light is weak, and one which is heavy is strong. If the design is the same, it is difficult to understand why the light ones do not break when the heavy ones do. Probably the lighter ones did not break because they were made of better material. It is also possible that the light carriers worked better because they were better proportioned.

On the Chicago Drainage Canal, with carriage speeds of about 800 ft. per min., the repair costs and loss of time, due to breakages, were serious. It seemed impossible to secure the buttons effectively to the button rope.

When the Lidgerwood Company undertook to build the thirteen cableways for handling concrete at Panama, no part of the construction was given more attention than the buttons and fall-rope carriers, for, under the terms of the contract, it was necessary to run the cableways at 1 850 ft. per min. These carriers, which for so many years had been the chief source of delays on cableways, were taken up more seriously than before, with the determination to solve the problem of a high-speed fall-rope carrier. The best carriers and the best buttons that brains and money could then devise were constructed and set up at the Lidgerwood Company's testing station. At 1 100 ft. per min. they collapsed, although the buttons held fast. All kinds of the toughest metals were used, and yet after weeks of experiment it was impossible to obtain anything that would withstand the shock of the

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button at a speed in excess of 1 150 ft. per min. Finally, after weeks of trial and continual rebuilding, the shock-absorbing principle was invented. The head of the fall-rope carrier was hinged instead of being rigidly connected to the body of the carrier. The button collided with a swinging eye instead of an elongated slot, and, instead of colliding at two points on opposite sides of the button, collided almost throughout its entire diameter.

As soon as this construction had been set in motion, a speed of 3 000 ft. per min. was developed, and the old law, that the impact was in proportion to the weight and increased as the square of the velocity, was emphasized in an unusual manner.

What the authors' conclusions would have been had the new shock-absorbing carrier and carriers been used is not difficult to anticipate.

The cableway engines used did not embody the numerous improvements now in use by the manufacturers of those engines, such as improved cylinder design and construction, larger, heavier, and better balanced reciprocating parts, etc., all resulting in higher speed and efficiency.

The obsolete buttons which the authors refer to were secured by a through pin and bearing in solder or Babbitt. The new buttons are taper-bored, the strands of the rope being opened and a tapered pin inserted. The taper of the bore and the pin is the same, and is less than the angle of repose. No other pin is used. The more the carriers hammer the buttons, the tighter the fastening. At Panama heavy jacks were necessary to remove the buttons after they had been in service for several months. The new buttons do not slip, and do not injure the button rope. Button ropes now frequently last four and five years. Therefore, the necessity for a divided button is not apparent.

The official reports at Panama show that these cableways at Gatun were kept at an efficiency of 99 per cent. Expressed in other terms, the entire cableway was ready for operation 99 min. out of every 100 they were called on for service, during the period recorded, and that applies not only to the fall-rope carriers, but to every part of the cableway. The efficiency of the new fall-rope carriers, therefore, can confidently be stated to be even in excess of 99 per cent.

On page 2275,* the authors state:

"A cableway is not satisfactory in handling excavated materials, as it is not only expensive, but slow."

They also state:

"The writers are of the opinion that, as a rule, the cableway should be restricted to moving plant, derricks, forms, and general work, and also such concreting as is advantageous to place with it, and that it

* *Proceedings*, Am. Soc. C. E., for September, 1914.

should not be made the main method for the transportation of all materials on the job, as it was here." Mr. Miller.

Among those who use cableways extensively for concreting operations, it is a common expression that the cableway is worth its investment merely for the purposes of moving plant, forms, etc., but the experiences of the Isthmian Canal Commission with cableways at Gatun Locks, Panama, by the John F. Casey Company, at Aspinwall, Pa., and Cleveland, Ohio, and by the H. E. Talbott Company, Grand Mére, Que., Canada, hardly bear out the conclusions of the authors.

The authors state that the record for a 10-hour run with the cableways was seldom more than 400 cu. yd. Does this mean 400 yd. for two cableways? At Panama runs as high as 60 cu. yd. per hour, with one cableway handling 2-yd. batches, were recorded. The John F. Casey Company, on thin section baffle walls, 6 ft. 9 in. wide at the bottom and 2 ft. wide at the top, has laid as much as 692 cu. yd. per day with one cableway. The H. E. Talbott Company placed, with one cableway, from 320 to 350 cu. yd. of concrete, with a travel of 1 100 ft., in 10 hours. Some extraordinary records are being made at Cleveland, Ohio, at present, by the John F. Casey Company. No other device will do what the cableway is adapted for, namely, handling excavation, placing concrete and large rock, moving forms, timber, derricks, and miscellaneous material, such as ironwork, etc. If time was limited, there is no reason why three or four cableways should not have been used.

In dam construction, also, there is nearly always the menace of high water during a part or all of the construction period. No device presents such effective insurance against that danger as the cableway. It stands safe throughout, and for removing auxiliary plant and material quickly when occasion demands, it is always ready. The speaker believes that had the latest type of high-speed cableways been used on this work, the authors would have modified their deductions.

W. E. Mitchell,* Esq. (by letter).—Referring to the discussion by Mr. R. D. Coombs, on this subject, Mr. Coombs, of course, did not know, what the authors of this paper knew, namely, that it is the intention to have a paper, discussing the purely electrical side of the development in Alabama, written by the Electrical Engineer for the American Institute of Electrical Engineers. When this paper comes out it will answer a number of the points raised by Mr. Coombs. Mr. Mitchell.

It may be stated here, however, that probably more attention than usual was given to the subject, not only of the transmission line voltage, but of the style and size of lines for both transmission and distribution. These matters were gone into very thoroughly, following

* Operating Mgr., Alabama Power Co., Birmingham, Ala.

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a careful survey of all high-tension systems in operation at the time, with the definite idea of making the service as reliable as possible.

Following out this idea, Birmingham, which of course is the main distributing point, is supplied by three 110 000-volt circuits, two by one route going practically direct to the city limits, and the third by an entirely separate route, so that the likelihood of interruptions due to severe storms in one locality will be minimized.

As stated in the paper, the transmission-line voltage is 110 000 volts, and the distribution lines are practically all of 22 000 volts. The middle cross-arm should have been given as being 5 ft. longer than the top and bottom cross-arms, making the offset of the middle wire 2 ft. 6 in.

The writer cannot agree with Mr. Coombs' statement that the high towers will not permit of the longer spans in rough country. There are times, of course, when considerably shorter spans are made necessary by intervening hills, but in general it was found in Alabama that with Type A towers the average span was 700 ft., and that in very few cases did it drop below this, although it was frequently as much as 750 or 800 ft.

The Type C towers, which, as a matter of fact, were used in the flat country, were not purposely designed for this work, but with the idea that steel-core aluminum would be used on their lines, as this called for less sag and for anchor towers at least every tenth tower. After the towers were ordered, however, it was decided to use copper, and hence the spacing had to be reduced. In general, a higher tower would be more economical in flat country. The Type C tower was designed light on purpose, as it would have been impossible to have anchored the steel-core aluminum at every tower, and it was the idea to anchor it at every tenth tower on an extra heavy Type B tower.

In regard to the specifications covering clearance and loading, a very careful investigation of the climatic conditions in Alabama was made, and it was found that only about three times in the last 20 years had conditions been such that sleet would have been formed; therefore it is thought that the use of $\frac{1}{4}$ in. of ice and 8 lb. of wind pressure was sufficiently conservative. The standard tower was used on all spans, as it was found to be still well within safe limits. Excessively long spans were purposely avoided in order that there would be no special construction, always extremely expensive. The only special towers used on the whole work were four for two river crossings, and the dead-end towers at each sub-station.

Referring to the criticism of distribution lines built on wooden poles, this also received careful consideration, and investigation of the life of creosoted poles throughout the South was made. Long-leaf yellow pine is extremely plentiful in this district and it was found

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that poles treated with 12 lb. of creosote showed no deterioration after 20 years. These poles proved to be very much cheaper than steel, although for comparative purposes, the company has actually put in one 25-mile line with flexible towers. This ran up the cost, however, to about 15% more than the same line built on creosoted poles. The writer does not believe that the steel poles would have enough longer life than the creosoted poles to justify them.

The size of the conductors on the 22 000-volt lines varies from No. 4 to No. 00 copper. The standard spacing, using creosoted, long-leaf, yellow pine cross-arms, is 5 ft. horizontally and 3 ft. 6 in. vertically between conductors. This is believed to be a more liberal spacing than is ordinarily used.

The writer was on this development from the commencement of the work, and has had the opportunity to be in direct charge of the operating from the time it went into commission up to the present date. During the past lightning season there was considerable trouble from punctured insulators, and one more disk is now being added to all strings. The entire system is now being inspected, and all insulators are being tested with a megger. It is believed that this will eliminate much of the trouble in the coming lightning season. On the distribution lines, where a 35 000-volt pin insulator has been used, there has been practically no trouble whatever.

The generating plant has worked out fully up to the expectations, with regard to simplicity of design, and has proved a good station to handle, from the operating point of view.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EDWARD COOK BURNS, M. Am. Soc. C. E.*

DIED OCTOBER 29TH, 1914.

Edward Cook Burns, the son of Barclay J. and Isabella Cook Burns, was born on January 30th, 1845, near the corner of Chambers Street and Broadway, in New York City, where his father was engaged in publishing and newspaper work. When the boy was three years old, his mother died, and he was taken to live with his grandmother at Jamestown, N. Y.

He received his early education in private schools at Jamestown, and was afterward sent to the Patterson Preparatory School in Detroit, Mich., where he was prepared for the Civil Engineering School of the University of Michigan from which he was graduated with the Class of 1868. Among his close friends and associates at the University were G. H. Benzenberg and the late Alfred Noble, Past-Presidents, Am. Soc. C. E., and Charles F. Brush, the inventor of modern electric lighting.

Mr. Burns had been employed during his summer vacations, from 1865 to 1867, as Assistant with the United States Lake Survey. In August, 1868, he was appointed Levelman on the Rockford, Rock Island and St. Louis Railway, remaining in that position until January, 1869, when he was made Division Engineer. From May to December, 1870, he was employed as Assistant Engineer on the Central Railroad of Iowa, and from April to July, 1871, as Division Engineer on the Burlington, Cedar Rapids and Minnesota Railroad.

Mr. Burns then returned to the East as Engineer in Charge of the extension of the Pittsburgh, Bessemer and Lake Erie Railroad until May, 1872, when he was appointed Assistant Engineer in charge of the construction of the Buffalo and Southwestern Railroad, from Gowanda to Jamestown, N. Y., under General Robert Ewing. From April, 1873, to August, 1874, Mr. Burns served as Acting Superintendent of the Jamestown Gas Light Company. In August, 1874, he returned to the Buffalo and Jamestown Railroad as Assistant Engineer, which position he retained until June, 1875.

From 1877 to 1885, he was employed as United States Assistant Engineer on river and harbor improvements at various places, including such important works as the construction of the Sault Ste. Marie

* Memoir prepared by the Secretary from information on file at the Society House.

Canal, the improvement of the St. Clair Flats Canal, the dredging of the Livingston Channel, etc., etc.

In 1885, Mr. Burns returned to Jamestown, N. Y., where, until his death on October 29th, 1914, after a year's illness, he was engaged in private practice as a Consulting Engineer. He was connected with many of the principal construction enterprises in Jamestown and the surrounding country, having served as Engineer of the Board of Public Works from 1894 to 1898. He was also one of the Conewango Swamp Drainage Commissioners and had charge of the construction of its line for the Chautauqua Traction Company. He was always in demand throughout the vicinity as an expert in the design and construction of highway and railroad bridges, and had planned and supervised the construction of the old and new Boatlanding Bridges and also the new Winsor Bridge, the latter two having been his last important work before ill-health compelled him to give up active practice.

On October 3d, 1872, Mr. Burns was married to Mary Graham, daughter of Major Thompson Graham, of Mercer, Pa., who, with one daughter, survives him.

Mr. Burns was a man of strong character and high ideals, who gave to his work his best thought and action. His strict integrity is shown in the character of his work, which was never slighted or cheapened to accomplish a temporary effect. He had a well-stored mind and, being fond of books and study, kept up with the advances and improvements of his profession. He was always considerate and courteous to every one, and will long be remembered by his friends and those who were closest to him for his pleasing personality, his kindly spirit, and his desire to add to the pleasures rather than the burdens of those with whom he was associated.

He was a member of the Delta Kappa Epsilon Fraternity and a Thirty-second Degree Mason.

Mr. Burns was elected a Member of the American Society of Civil Engineers on July 5th, 1882.

HURD CLARENCE HURD, M. Am. Soc. C. E.*

DIED JULY 8TH, 1914.

Hurd Clarence Hurd was born at Helena, Tex., on April 16th, 1870. The early years of his life were spent in a community in which the Spanish language was spoken to a considerable extent. He acquired a knowledge of this language at that time and afterward perfected it

* Memoir prepared by Wendell M. Reed, M. Am. Soc. C. E.

in high school and university, making much use of it throughout his professional career.

He was graduated with the degree of C. E., from Princeton University, in June, 1893. His first work was in connection with the Sewer Department of Washington, D. C. In 1897, he went with the Nicaraguan and Isthmian Canal Commissions as Instrumentman, but at the end of the first year of his engagement he became First Assistant to the Chief Hydrographer, holding this position until 1901. Although this work was interesting, and developed individual initiative, it was also fraught with hardships, and Mr. Hurd, besides experiencing the minor troubles, also suffered an attack of yellow fever. That he came out of this country not completely broken in health, was a strong testimonial of his rugged constitution and temperate manner of living.

In September, 1902, he was appointed a Special Agent of the State Department, in Nicaragua and Costa Rica. In 1903, he became attached to the United States Reclamation Service, and spent the next two years in charge of important surveys and construction of canals, dams, and irrigation systems in Arizona, California, New Mexico, and Montana.

In 1905, the call of the South again reached him, and he went to Peru as Chief of the Hydraulic Commission for the study of irrigation. The Peruvian Government having requested the recommendation of a suitable person for this position, Mr. Hurd was named by the United States Government.

In 1907 Mr. Hurd returned to the United States and took up hydro-electric work in California. He followed this line of work for five years and, during that time, had charge of the construction of several important hydro-electric developments. His skill in reconnoissance surveys and plans gave him such a standing that he was employed, in 1912, by the Ebro Irrigation and Power Company, to take charge of the location, design, and construction of a large lined power canal, aqueducts, siphons, etc., in Spain.

He was in charge of this work for about one year when the general financial depression compelled its cessation. Mr. Hurd then returned to the United States and was taking a much needed rest when he accidentally met his death by drowning in Chesapeake Bay.

Mr. Hurd was modest and unassuming in manner, but soon impressed those with whom he came in contact with his sterling worth, and professionally he ranked very high.

He was married in 1905 to Miss Grace Lewis, of Roswell, N. Mex., who, with four children, survives him.

Mr. Hurd was elected a Member of the American Society of Civil Engineers on July 2d, 1913.

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